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RECORD OF REVISIONS

Rev	Date	Description	POC	OIC
0	6/28/99	Initial issue (as FEM)	Doug Volkman, *PM-2*	Dennis McLain, *FWO-FE*
1	2/09/04	Changed FEM to ESM; Incorporated IBC & ASCE 7 in place of UBC 97; Incorporated DOE-STD-1020-2002 in place of DOE-STD-1020-94; Incorporated concepts from DOE O 420.1A	Mike Salmon, *FWO-DECS*	Gurinder Grewal, *FWO-DO*

RESPONSIBLE ENGINEERING STANDARDS POC AND COMMITTEE

for upkeep, interpretation, and variance issues

PC-1 and PC-2	**Structural POC/Committee.**

II PC-1 & PC-2 DESIGN & ANALYSIS REQUIREMENTS

A. This Section provides requirements for the design and analysis of PC-1 and PC-2 building structures (Subsection II.1.0), nonstructural components, equipment, distribution systems, and non-building structures (Subsection II.2.0). Structural design requirements generally consist of the following:

- Establish structural arrangement/geometry
- Establish loads and load combinations
- Evaluate the structural response to the loads
- Specification of structural capacity and drift limits (acceptance criteria)
- Special design considerations, such as ductile detailing requirements

B. Following the graded approach philosophy outlined in DOE Order 420.1A, "Facility Safety," and implemented in guidance document, DOE G 420.1-2, and design/evaluation standard, DOE-STD-1020, PC-1 and PC-2 structures, systems, and components are to be designed following the provisions of the International Building Code (IBC). The IBC refers to ASCE 7 (Minimum Design Loads for Buildings and Other Structures) for structural loads, wind evaluation, and most seismic design/evaluation requirements. In some instances, the IBC amends the ASCE 7 seismic criteria with modifications. Hence, the primary source documents for design of PC-1 and PC-2 structures, systems, and components are the IBC, ASCE 7, and material standards (i.e., ACI, AISC) as referenced in ASCE 7.

1.0 PC-1 & PC-2 BUILDING STRUCTURES

1.1 STRUCTURAL LOADS

A. Loads listed in Table II-1 shall be considered in the design of PC-1 and PC-2 building structures at LANL. Criteria for assessment and mitigation for other natural phenomena loads listed in Appendix C of DOE O 420.1A, but not addressed in Section II (e.g., volcanic events, lightning, forest fires, drought, fog, frost, extreme temperatures, etc) must be developed on a site-specific basis per DOE G 420.1-1. LANL FWO-Design Engineering & Construction Services (DECS) is currently developing design bases for these other loads.

B. Each of the loads listed in Table II-1 is discussed in more detail in the remainder of this section and minimum loads to be used for design of structures are defined. Wind, snow, and earthquake loads appropriate for LANL are specified herein. In addition, there are LANL specific loads beyond those specified in the building code that are defined in this section.

1.1.1 Dead Load (D)

A. Dead loads consist of the weight of all materials of construction incorporated into the building including walls, floors, roofs, ceilings, built-in partitions, finishes, cladding, and other similar architectural and structural items. Also, fixed service equipment weight including that of cranes comprise dead load.

B. Best estimates of the actual weights comprising the dead load shall be used in design. 10 psf shall be added to the best estimate dead load for all floors for use in design to accommodate future dead load.[1]

Table II-1 Design Structural Loads

Loads from the IBC and ASCE 7	Loads specific for LANL structures
Dead load	Experimental blast loads
Live load and roof live load	Accidental blast loads
Wind load	
Snow load	
Rain load	
Self straining forces	
Load due to fluids	
Lateral soil pressure loads	
Flood loads	
Earthquake loads	

1.1.2 Live Load (L) and Roof Live Load (L_r)

A. Live loads (L) are those loads produced by the use and occupancy of a building or other structure and do not include construction or environmental loads as wind load, snow load, rain load, earthquake load, flood load, or dead load. The live load to be used in design shall be determined following the provisions of ASCE 7 [11]. Live load provisions include uniformly distributed loads, concentrated loads, impact loads, reduction in uniformly distributed live loads, and crane operating loads.

B. Live loads used in the design of buildings shall be in no case less than the minimum uniformly distributed loads required in Table II-2. Floors or other similar surfaces shall be designed to support safely the uniformly distributed load or the concentrated load given in Table II-2, whichever produces the greater load effects. Unless otherwise specified, the indicated concentrated load shall be assumed to be uniformly distributed over an area of 2.5 feet square and shall be located so as to produce the maximum load effects in the structural members.

C. Live loads on a roof (L_r) are those produced (1) during maintenance by workers, equipment, and materials; and (2) during the life of the structure by movable objects such as planters and by people. All roofs at LANL shall be designed for a minimum roof live load of 30 psf.[2]

[1] The additional applied 10 psf load is a conservative additional LANL specific requirement. The 10 psf future dead load need not be included in the renovations or modifications to existing structures, if an accurate compilation of existing dead load is conducted based on documented site specific verification of the load.

[2] The 30 psf minimum roof live load at LANL is specified to cover potential overloads caused by snow, rain on snow, and maintenance activities, and is based on judgment and history.

1.1.3 Wind Loading (W)

A. Wind loading (W) shall be calculated using the procedure prescribed in Chapter 6 of ASCE 7 [11]. Wind loads are external and internal pressures acting on all surfaces of the building. External pressures on windward walls are positive in sign signifying pressure acting toward the wall surface.

B. External pressures on leeward walls or the roof are negative in sign signifying pressure acting away from the wall or roof surface (i.e., suction). Internal pressures are considered in partially enclosed or enclosed buildings and must be considered to act both toward and away from the internal surfaces. Wind pressures are determined from the product of the velocity pressure, q_z, a gust effect factor, G, and force (drag) coefficient, C_f. Velocity pressure is a function of the basic wind speed, V, specified for building design.

Table II-2 Minimum Live Load[3]

Use	Uniform, psf	Concentrated, lbs[4]
Access floor systems		
Office use	50	2000
Computer use	100	2000
Assembly areas and theaters		
Fixed seats (fastened to floor)	60	
Lobbies	100	
Movable seats	100	
Platforms (assembly)	100	
Stage floors	150	
Corridors – first floor	100	
Manufacturing and Laboratories		
Light	125	2000
Heavy	250	2000
Office Buildings		
Lobbies and first floor corridors	100	2000
Offices	50	2000
Corridors above first floor	80	2000
Storage Warehouses		
Light	125	
Heavy	250	
Roof (LANL specific minimum requirement)	30	

[3] Loads are for typical LANL usage. See Table 4-1 of ASCE 7 for additional minimum loads.

[4] Applied over a 2.5 feet square area

C. The basic wind speed, V, used for determination of design wind loads on buildings and other structures from DOE-STD-1020 is given in Table II-3 for PC-1 and PC-2. The basic wind speed is defined as a 3 second gust speed at 33 feet above the ground in exposure Category C per ASCE 7.

D. Velocity pressure, q_z, evaluated at height z shall be calculated by the following equation:

$$q_z = 0.00195 * K_z * K_{zt} * K_d * V^2 * I \ (\mathrm{lb/ft^2; V\ in\ mph}) \qquad \text{Eq. II-1}$$

E. Equation II-1 differs from Equation 6-15 of ASCE 7 because of adjustments for the altitude and temperature conditions at LANL. The constants in ASCE 7 reflect the mass density of air for the standard atmosphere, i.e., temperature of 59°F and sea level pressure. The above equation used LANL site conditions from Ref. 33 for elevation of 7380 feet and air temperature of 48.1°F.

Table II-3 Design Wind Speeds

Performance Category	Average Return Period	Basic Wind Speed, V	Importance Factor, I	Exposure
PC-1	50 years	90 mph	1.0	C
PC-2	100 years	96 mph	1.0	C

F. Disregard shelter from changes in the ground elevation and the height of other buildings by using Exposure C.[5] The importance factor (I) for PC-1 and PC-2 structures for wind loading is 1.0 per DOE-STD-1020 [1].

G. The factor K_{zt} in Equation II-1 is a topographic factor that accounts for wind speed-up effects at isolated hills, ridges, and escarpments constituting abrupt changes in the general topography. The mesas at LANL that are preferred building sites due to flood considerations may be subject to wind speed-up effects. These effects shall be considered by using Section 6.5.7 and Figure 6-4 of ASCE 7.

1.1.4 Snow Loading (S)

A. Snow loading (S) used for roof load in structure design shall be calculated using the procedure prescribed in Chapter 7 of ASCE 7 [11]. The ground snow load, p_g, for PC-1 and PC-2 structures at LANL is based on a site-specific analysis, as shown in Table II-4. The snow loadings for PC-1 and PC-2 buildings have been derived from a statistical study of 77 years of LANL site-specific data. DOE-STD-1020 [1] indicates an NPH annual probability of exceedance for wind of 2 x 10^{-2} and 1 x 10^{-2} (50 year and 100 year return period) for PC-1 and PC-2 buildings, respectively, which is adopted and is appropriate for snow load. An additional 5 psf for rain on snow shall be used for relatively flat roof profiles (slopes of 0.5 vertical to 12 horizontal, or less).[6]

[5] Using exposure C is a conservative LANL specific requirement, in that it neglects the potential sheltering from other adjacent structures and trees etc.

[6] ASCE 7, Section 7.10.

B. The snow load must be factored for unbalanced accumulation of snow at valleys, parapets, roof structures and offsets in roofs of uneven configuration (drifts) as described in Chapter 7 of ASCE 7. The importance factor (I) for PC-1 and PC-2 structures for snow loading is 1.0 to be consistent with DOE-STD-1020 [1] wind load provisions. Note that the minimum 30 ps roof live load required by LANL will often be the controlling design live load for PC-1 and PC-2 structures.

Table II-4 LANL Basic Snow Loads

Performance Category	Average Return Period	Ground Snow Load, p_g
PC-1	50 years	16 psf
PC-2	100 years	19 psf

1.1.5 Rain Loading (R)

A. Rain loading (R) used for roof load in structure design shall be calculated using the procedure prescribed in Chapter 8 of ASCE 7 [11]. Roof drainage systems shall include primary drains or scuppers and secondary (overflow) drains or scuppers. The flow capacity of secondary drainage shall not be less than that of the primary drainage system. Each portion of the roof shall be designed to sustain the load of all rainwater that can accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage at its design flow. The rain load on the undeflected roof in psf is given by:

$$R = 5.2 * (d_s + d_h) \qquad \text{Eq. II-2}$$

where d_s is the depth of water in inches on the undeflected roof up to the inlet of the secondary drainage system when the primary system is blocked and d_h is the additional depth of water on the undeflected roof above the inlet of the secondary drainage system at its design flow.

B. Ponding refers to the retention of water due solely to the deflection of relatively flat roofs. Such roofs shall be investigated by structural analysis for the larger of rain or snow loads to ensure that they possess adequate stiffness to preclude progressive deflection (i.e., instability) when rain fails on them or meltwater is created from snow on them. The primary drainage system within an area subjected to ponding shall be considered to be blocked in this analysis.

1.1.6 Self Straining Forces (T)

A. The structural design shall consider self-straining forces arising from contraction or expansion resulting from temperature change, shrinkage, moisture change, creep in component material, movement due to differential settlements of foundations, or combinations thereof. Unless specifically addressed through analysis, the effects of self straining forces shall be accommodated by the placement of relief joints, suitable framing systems, or other details.

1.1.7 Fluid Loads (F)

A. Fluid load (F) is the load resulting from the pressure of the fluid. These loads are from fluids with well-defined pressures and maximum height (such as fluids in tanks).

1.1.8 Lateral Soil Pressure Loads (H)

A. Subterranean structural walls shall be designed to resist lateral soil pressure loads (H). These include basement walls, foundation walls, retaining walls, walls of underground vaults for storm or wastewater drainage or electrical junctions, and underground tanks. These loads are generally due to lateral pressure of soil and water in soil. Design lateral soil loads are given in Chapter 5 of ASCE 7 [11] or Section 1610 of the IBC [5] as a function of backfill soil material type. These loads are the minimum design loads unless specified otherwise by a soil investigation report approved by LANL.

B. Active pressure and at-rest soil pressure loads are given in ASCE 7 and the IBC. Basement walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure. Retaining walls free to move and rotate at the top are permitted to be designed for active pressure. Underground tanks having walls that can displace inward under lateral soil pressure loading shall be designed for soil pressure loads which take into account these lateral displacements. Design lateral pressure from surcharge loads shall be added to the lateral earth pressure load. When a portion or the whole of the adjacent soil is below a free water surface, lateral pressures shall be evaluated based on the weight of the soil diminished by buoyancy, plus full hydrostatic pressure. Design lateral pressure shall be increased if soils with expansion potential are present at the site based on criteria provided by a geotechnical expert for the project.

C. Lateral soil pressure loads also result from earthquake ground shaking per ASCE 7 Section 9.7.5.1. ASCE 4 [6], Section 3.5.3 provides acceptable means of accounting for seismic-induced lateral soil pressures on subterranean structural walls. Seismic-induced soil pressure loads are included in earthquake loads (E).

1.1.9 Flood Loads (F_a)

A. Information about LANL regional flooding and runoff analyses for local precipitation at individual LANL sites may be obtained through the LANL Chapter 3 (Civil) Point of Contact (POC). LANL shall re-evaluate this flood issue prior to any major project initiation for potential increases in flood level due to increases in area development and reductions in site drainage paths.

B. In accordance with DOE-STD-1020, PC-1 and PC-2 buildings are to be designed for flood hazards at mean return periods of 500 and 2000 years, respectively. DOE-STD-1020 specifies that both regional flood hazards and local precipitation must be considered. Regional flood hazards arise from river overflow, dam failure, or levee/dike failure. LANL mesa top facilities would typically not be subject to regional flood hazards. However, all sites must be designed for the effects of intense local precipitation.

C. The procedure presented in DOE-STD-1020 for design and evaluation for local precipitation is to develop an initial site drainage system (and roof drainage system) for not less than the 25 year, 6 hour storm. The next step is to perform a hydrological analysis for the site based on the characteristics of the site and the site drainage system to establish the level of local flooding around the facility. For this evaluation, the return period rainfall for the appropriate PC level is used. The design of the site drainage system is addressed in the Civil chapter of the ESM. The hydrological analysis to determine the resulting water levels from local precipitation and the assessment as to whether those water levels are acceptable is also covered in the ESM Civil chapter.

D. The evaluation of the structure for the hydrostatic water loads at the resulting water depths plus any hydrodynamic loads is covered by this Structural chapter of the ESM. Flood loads shall include both hydrostatic and hydrodynamic loads. Design loads used for structures located in areas prone to flooding due to either regional flooding or during runoff from local precipitation shall follow the provisions in Chapter 5 of ASCE 7 [11]. Where structures are subject to flood loads, structural systems of buildings or other structures shall be designed, constructed, and anchored to resist flotation, collapse, and permanent lateral displacement due to action of flood loads associated with the design flood.

1.1.10 Earthquake Effects (E)

A. PC-1 and PC-2 building structures shall be evaluated and designed for earthquake ground shaking primarily using the requirements of ASCE 7 [11] augmented with LANL site specific seismic input. Earthquake effects (E) are the forces, stresses, or deformations in structural members as determined from seismic analysis conducted by either equivalent static or dynamic analysis methods. Earthquake loads are given in terms of seismic base shear and distributed lateral forces determined from Sections 9.5.4 or 9.5.5 of ASCE 7 (Simplified Analysis Procedure or Equivalent Lateral Force Procedure) or the seismic base shear and distributed seismic inertial forces determined from a dynamic seismic analysis per Sections 9.5.6, 9.5.7, or 9.5.8 of ASCE 7 (Modal Analysis Procedure, Linear Response History Analysis Procedure, or Nonlinear Response History Analysis Procedure). The dynamic seismic analysis would generally follow the response spectrum method (per Section 9.5.6 of ASCE 7) but could also be performed by time history methods, using a response spectrum compatible time history of earthquake accelerations. By either equivalent static or dynamic analysis methods, the earthquake load is based on the 5 percent damped input ground response spectrum appropriate for the site at the ground surface.

B. Earthquake loads determined by ASCE 7 seismic design methodology are based on the following design spectral response parameters.

- S_{DS} – 5% damped design spectral response acceleration at short periods (0.2 sec.)
- S_{D1} – 5% damped design spectral response acceleration at one second period

C. Following ASCE 7, these parameters are evaluated for the LANL site from the mapped spectral accelerations at periods of 0.2 and 1 second for a rock site (Site Class B) at 2% probability of exceedance in a 50 year building life (2500 year average return period). From the maps in ASCE 7 and from the 1996 mapped values in the USGS web site, the 2500 year mapped rock spectral accelerations at LANL (35^{o} 52'N and –106^{o} 19'W) are:

- $S_S = 0.60g$
- $S_1 = 0.19g$

D. Continuing with the ASCE 7 approach, the maximum considered earthquake spectral response accelerations, S_{MS} and S_{M1}, are equal to the above values for a rock site but are adjusted by F_a and F_v factors for other site soil conditions. LANL site conditions are typically considered to be Site Class D, which is a stiff soil profile where average shear wave velocities over the top 100 feet are between 600 and 1200 feet per second. Based on ASCE 7 Tables 9.4.1.2.4a and 9.4.1.2.4b, respectively, for Site Class D and the mapped accelerations given above, the factors F_a and F_v are 1.32 and 2.04. Therefore, the maximum considered earthquake spectral response accelerations from ASCE 7 are:

- $S_{MS} = F_a * S_S = 1.32 * 0.60 = 0.79g$ (ASCE 7 Values)
- $S_{M1} = F_v * S_1 = 2.04 * 0.19 = 0.39g$

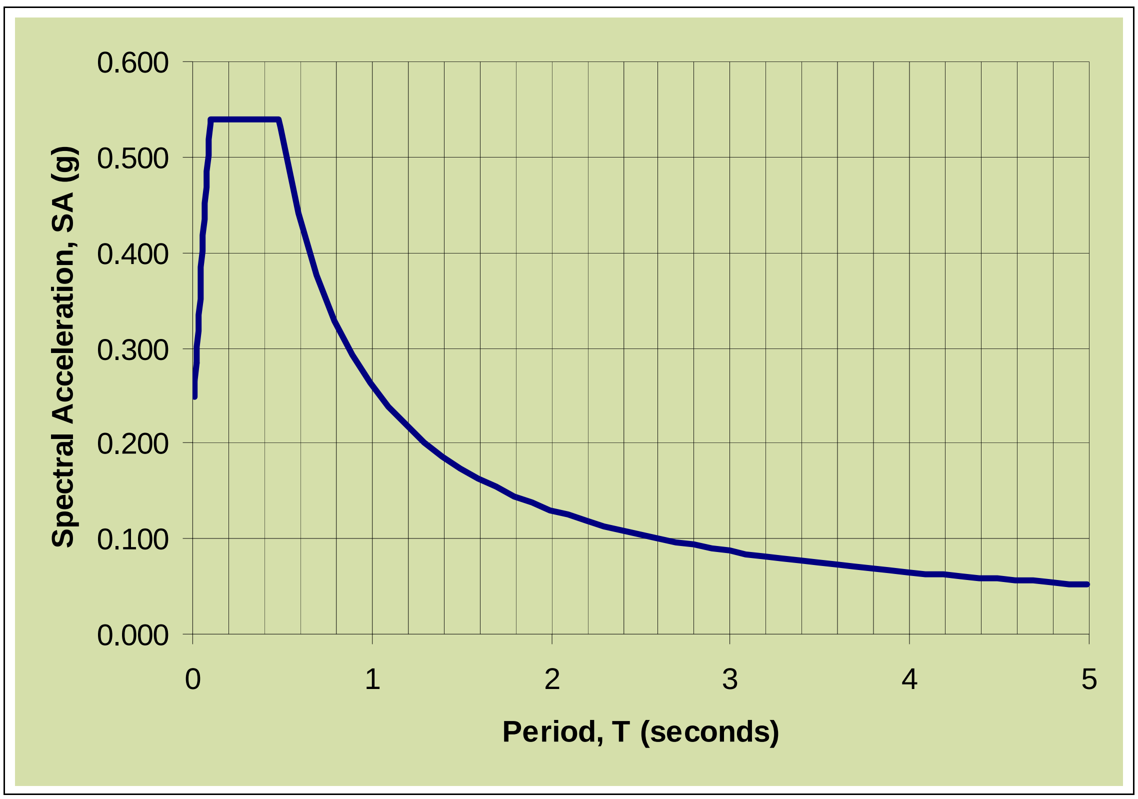
E. The above values are the 2500-year return period ground spectral accelerations for Site Class D soil conditions. These values may be compared to LANL site-specific ground response spectra determined from the LANL seismic hazard study. The peak of the LANL site-specific ground response spectra for 2500 years is about 0.81g and the spectral value at one-second period is 0.39g, both very close to the IBC values. The LANL site-specific values will be used for the design of PC-1 and PC-2 structures, systems, and components. Hence:

- S_{MS} = 0.81g (LANL Specific Values)
- S_{M1} = 0.39g

F. Per ASCE 7 Section 9.4.1.2.5, the design spectral response accelerations are equal to:

- S_{DS} = (2/3) * S_{MS} = 0.54g
- S_{D1} = (2/3) * S_{M1} = 0.26g

G. These design spectral response accelerations are used directly in the simplified seismic analysis procedure of ASCE 7 Section 9.5.4 or the equivalent lateral force procedure of ASCE 7 Section 9.5.5 to establish earthquake loads. When seismic analyses are performed by the modal analysis procedure of ASCE 7 Section 9.5.6, earthquake input motion is defined in terms of a 5% damped ground response spectrum. The design ground response spectrum shall have the shape shown in ASCE 7 Figure 9.4.1.2.6 with the amplitude based on the design spectral response accelerations, S_{DS} and S_{D1}. The design ground response spectrum for LANL PC-1 and PC-2 structures is shown in Figure II-1.



LANL PC-1 and PC-2 Design Response Spectrum

H. For the linear response history analysis procedure of ASCE 7 Section 9.5.7, modified recorded or synthetic time histories that are compatible with the input ground response spectrum described above shall be used. Rules for creating spectrum compatible time histories as given in ASCE 4 [6] shall be followed. A single time history may be used for two-dimensional analyses and a set of two uncorrelated horizontal time histories may be used for three-dimensional analyses. Rules for developing uncorrelated time histories in ASCE 4 shall be followed. Note that ASCE 7 requires the use of multiple time histories. Exception to this requirement is taken herein because of the stringent spectrum matching requirements in ASCE 4. For the nonlinear response history analysis of ASCE 7 Section 9.5.8, a suite of not less than three recorded or modified recorded time histories shall be used following the requirements in ASCE 7 Sections 9.5.7.2 and 9.5.8.2. By these requirements, actual recorded time histories from events having magnitudes, fault distances, and source mechanisms controlling the earthquake ground motion at LANL are required. Information on controlling earthquakes may be determined from the LANL seismic hazard evaluation. Recorded time histories at various magnitudes and distances may be obtained from the time history database in NUREG/CR-6728.

I. Vertical earthquake ground motion is not explicitly considered in the seismic evaluation of PC-1 and PC-2 structures. The vertical earthquake load is implicitly included in ASCE 7 provisions by adding a scaled dead load case where the scale factor is 0.2 times S_{DS}, as shown below.

J. Earthquake effects (E) are combined with the effects of other loads in accordance with the load combinations presented later in this section. Again, earthquake effects are the seismic response forces, stresses, or deformations in structural elements as determined from equivalent static or dynamic seismic analyses. Per ASCE 7, E is defined by:

$$E = \rho * Q_E \pm 0.2 * S_{DS} * D \qquad \text{Eq. II-3a}$$

$$E = \Omega_o * Q_E \pm 0.2 * S_{DS} * D \qquad \text{Eq. II-3b}$$

where Q_E is the seismic response in structural elements due to horizontal earthquake ground motion defined by the lateral forces or response spectrum given above and D is the response in structural elements due to dead load. ρ is a reliability coefficient based on the extent of structural redundancy present in a building as defined in ASCE 7 Section 9.5.2.7 and Ω_o is an overstrength factor as defined in ASCE 7 Table 9.5.2.2. The second term for E (Eq. II-3b) is used where special seismic loads are required by ASCE 7 or for certain steel structural components (primarily connections) specified in AISC, "Seismic Provisions for Structural Steel Buildings" [18].[7]

[7] This load designation accounts for the fact that some elements of properly detailed structures are not capable of safely resisting ground shaking demands through inelastic behavior. Examples include collector elements or some connections in steel structures.

1.1.11 Designed Experiment Blast Load (L_{EB})

A. LANL conducts experiments involving explosions. When such experiments take place within a building, the experimental explosion effects shall be contained within an internal structure such that loadings on the building are minimized. The design of the internal containment structure is not within the scope of this chapter. However, the containment structure may impose reaction forces on the building during the experimental explosion. Such reaction forces are one form of designed experiment blast loads, L_{EB}. In addition, experimental explosions may take place exterior to the building under consideration.

B. Blast effects on the building from such exterior experimental explosions are another form of designed experiment blast loads, L_{EB}. The forcing function and duration of the blast loading shall be based on the TNT equivalency of the maximum quantity and closest possible distance from the structural component in question of explosives and propellants in the designed experiment in accordance with TM-5-1300 [22] or reaction forces from the design of the internal containment structure provided by LANL. The dynamic characteristics of these short duration blast loads shall be considered in building evaluation and design. For external explosions, potential fragments and ground shock shall be considered in addition to blast overpressure.

1.1.12 Accidental Blast Load (A_B)

A. Permanent explosives facilities shall comply fully with TM 5-1300 [22] and DOE/TIC-11268 [32]. Blast resistant design for personnel and facility protection shall be based on the TNT equivalency of the maximum quantity of explosives and propellants. In accordance with TM 5-1300 [22], the TNT equivalency shall be increased by 20% for design purposes. *Accidental explosions might also result from the storage and handling of flammable materials, such as hydrocarbons. A release of flammable vapor in a region of adequate confinement and obstacle density is a potential source of a vapor cloud explosion. The blast load resulting from a potential vapor cloud explosion, in terms of incident side-on overpressure and the associated impulse or duration may be estimated using the CCPS book, "Guidelines for Evaluating the Characteristics of Vapor Cloud Explosions, Flash Fires, and BLEVEs" [45].* Blast load effects shall be provided by LANL.

B. The design of all new facilities, or those with major modifications, shall conform to the DOE Explosives Safety Manual, DOE 440.1-1 requirements for either accidental explosions of explosives or vapor cloud explosions. Protective construction design features are provided in TM 5-1300 and DOE/TIC-11268. When evaluating for accidental blast load, the loading A_B shall replace E (earthquake) loads in the load combination equations. All potential blast effects shall be considered including blast overpressure, gas pressure, fragments, and ground shock.

1.2 LOAD COMBINATIONS

A. Buildings and other structures shall be designed by either Strength Design (SD), Load and Resistance Factor Design (LRFD), or Allowable Stress Design (ASD) for all of the loads described above, except for accidental blast loads. Specific criteria for low probability, high amplitude, very short duration accidental blast loads based on structural response well into the nonlinear range are specified in this Chapter. A single design approach shall be used exclusively for proportioning elements of a particular construction material throughout the structure.

1.2.1 Combinations for Strength Design (SD) or Load & Resistance Factor Design (LRFD)

A. Structures, components and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the combinations presented in Table II-5. Each limit state shall be investigated with the effects of one or more loads not acting (except for Dead Load) when they possibly counteract each other. The load factors and combinations reflect the structural demand uncertainty from the individual loads and their combined effect to potential structural failure regardless of the structural material used in the design. Differences in material reliability are accounted for while determining the structural capacity with capacity reduction factors unique to the particular material and stress state.

Table II-5 Load Combinations for Strength Design, LRFD, or Accidental Blast Design

1.	**$1.4 (D + F)$**
2.	**$1.2 (D + F + T) + 1.6 (L + H) + 0.5 (L_r$ or S or $R)$**
2a	**$1.2 (D + F + T) + 1.6 (L + H) + 0.5 L_r + L_{EB}$**
3.	**$1.2 (D + F) + (1.6 L_r$ or $1.6 S$ or $1.6 R) + (0.5 L$ or $0.8W)$**
3a	**$1.2 (D + F) + 1.6 L_r + 0.5 L + L_{EB}$**
4.	**$1.2 D + 1.6W + 0.5 L + 0.5 (L_r$ or S or $R)$**
4a	**$1.2 D + (0.8W + 1.0F_a) + 0.5 L + 0.5 (L_r$ or S or $R)$**
5.	**$1.2 D +1.0 E + 0.5 L + 0.2 S$**
6.	**$0.9 D + 1.6 W + 1.6 H$**
6a	**$0.9 D + (0.8W + 1.0F_a) + 1.6 H$**
7.	**$0.9 D + 1.0 E + 1.6 H$**
8.	**$D + A_B + H + 0.5 L + 0.2 S$**
Notes:	
a)	**Load factor for L for these Combinations 3, 4, and 5 shall equal 1.0 for garages, areas occupied as places of public assembly and all areas where L is greater than 100 psf.**
b)	**H are normally acting earth pressure loads (not due to earthquake)**
c)	**The load factor on H shall be set equal to zero in Combinations 6 and 7 if the structural action due to H counteracts that due to W or E or A_B. Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance.**
d)	**In areas where flood loads must be considered for structural design, Load Combinations 4a and 6a shall also be considered.**
e)	**In locations of the building where engineered blast loads must be considered for structural design, Load Combinations 2a and 3a shall also be considered.**
f)	**For Load Combinations 6, 6a, & 7, D shall not include the LANL specified 10psf**

B. The blast loadings that are considered at LANL are not included in ASCE 7 load combinations. Therefore, the ASCE 7 load combinations were modified to include the blast loadings. Engineered blast load, L_{EB} will most likely affect only limited portions of the building when experiments are conducted. Because these loads can be repeated many times during the life of the structure, the structure should be designed to remain elastic. However, the dynamic characteristics of very short duration blast loads should be accounted for in the structural design.

C. Accidental blast loads, A_B, are infrequent events. These events do not have the same level of recurrence data as is available for natural phenomena hazards. Site specific data has not been evaluated. In lieu of quantitative information, it is judged that the probability of accidental blasts is comparable to the frequency of occurrence of PC-3 and PC-4 earthquakes (i.e., less probable that PC-1 and PC-2 earthquakes). Therefore, load combinations have been specified to be consistent with this judgment.

D. Where flood loads need to be considered in structural design, additional Load Combinations 4a and 6a where 1.6W in Combinations 4 and 6 is replaced by 0.8 W + 1.0 F_a shall be considered. The most unfavorable effects from both wind and earthquake shall be investigated, where appropriate, but they shall not be considered to act simultaneously.

1.2.2 Combinations for Allowable Stress Design (ASD)

A. Structures for which capacities are determined in accordance with Allowable Stress Design (ASD) shall be designed for the worst case of the combinations presented in Table II-6. Effects of one or more loads not acting (except for Dead Load) shall be considered. The calculated stresses based on the load combinations in Table II-6 shall be equal or less than the allowable stresses based on the applicable material design code and as noted below.

B. The load combinations given in Table II-6 are to be used without the separate 1/3 increase allowance for wind and seismic loadings given in past ASD codes. A December 2001 supplement to the AISC ASD code for steel structures [12] removes the 1/3 increase in allowable stress specification. The ASD provisions in Chapter 2 of the Masonry Code [17]. currently allows the 1/3 increase in allowable stress but has different load combination equations. For masonry, the load combinations of Table II-6 are to be used in combination with normal material allowable stress limits without the 1/3 increase for wind and seismic.

C. Based on the design earthquake shaking levels presented in Section II.1.1.10, structural design requirements must conform to those for Seismic Design Category D per ASCE 7 and IBC seismic provisions (see Section II.1.3). For this Seismic Design Category, ASD is not permitted for design of the lateral force resisting members of steel structure per ASCE 7 Section A.9.8.4 and IBC Section 2205.2.2.

Table II-6 Load Combinations for Allowable Stress Design

1.	**$D + F$**
2.	**$D + H + F + L + T$**
2a	**$D + H + F + L + T + L_{EB}$**
3.	**$D + H + F + (L_r \text{ or } S \text{ or } R)$**
4.	**$D + H + F + 0.75(L + T) + .75\ (L_r \text{ or } S \text{ or } R)$**
4a	**$D + H + F + 0.75(L + T) + .75\ L_r + L_{EB}$**
5.	**$D + H + F + (W \text{ or } 0.7\ E)$**
5a	**$D + H + F + W + 0.75F_a$**
6.	**$D + H + F + 0.75(W \text{ or } 0.7\ E) + 0.75\ L + 0.75(L_r \text{ or } S \text{ or } R)$**
6a	**$D + H + F + 0.75(W + F_a) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$**
7.	**$0.6\ D + W + H$**
7a	**$0.6\ D + W + 0.75F_a + H$**
8.	**$0.6\ D + 0.7\ E + H$**
NOTES:	
a)	**L_{EB} shall replace L for those limited portions of the structure where it is greater than L.**
b)	**In areas where flood loads must be considered for structural design, Load Combinations 5a, 6a, and 7a shall also be considered.**
c)	**In locations of the building where engineered blast loads must be considered for structural design, Load Combinations 2a and 4a shall also be considered.**
d)	**For Load Combinations 7, 7a, & 8, D shall not include the LANL specified 10psf**

1.3 STRUCTURAL DESIGN

A. Buildings shall be designed in accordance with strength design, load and resistance factor design, or allowable stress design, as permitted by the applicable material standards. Buildings shall be designed to support safely the loads in load combinations defined above without exceeding the appropriate limits for the given materials of construction. In addition, structural systems and members thereof shall be designed to have adequate stiffness to limit deflections and lateral drift for comfort of building occupants, for serviceability of the building, and to limit structural damage in the case of occasional loads such as earthquake.

B. Load effects on structural members and their connections shall be determined by methods of structural analysis that take into account equilibrium, general stability, geometric compatibility and both short- and long-term material properties. Any system or method of construction to be used shall be based on a rational analysis in accordance with well-established principles of mechanics. Such analysis shall result in a system that provides a complete load path capable of transferring loads from their point of origin to the load-resisting elements.

C. The total lateral force due to earthquake, wind, or blast pressure shall be distributed to the various vertical elements of the lateral force resisting system in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm. Provisions shall be made for increased forces induced on resisting elements of the structural system due to eccentricity between the center of application of the lateral forces and the center of rigidity of the lateral force resisting system. Every structure shall be designed to resist the overturning and/or sliding effects caused by lateral forces from wind, lateral soil loads, earthquake, and blast pressure.

D. By DOE-STD-1020, PC-1 and PC-2 buildings shall be considered to be in Seismic Use Groups I and III, respectively. Based on these Seismic Use Groups and the seismic input parameters for LANL of S_{DS} of 0.54g and S_{D1} of 0.26g as evaluated above, PC-1 and PC-2 structures at LANL shall be designed for earthquake loads following the ASCE 7 and IBC provisions for Seismic Design Category D.[8]

E. The IBC and ASCE 7 specify importance factors for snow load, wind load, and seismic load. The snow and wind load importance factors have the effect of increasing the applied snow and wind load. The seismic importance factor has the effect of reducing the response modification factor, R (to be defined later in the chapter), which in turn, enhances post-earthquake operability and function. DOE-STD-1020 specifies that the importance factor for wind shall be 1.0 for both PC-1 and PC-2, with increased wind speed differentiating these performance categories. Wind speeds at a lower probability of exceedance are used for PC-2 compared to that for PC-1. This is a different approach than is employed in the IBC and it will be used for LANL structural design. Snow load will be treated in the same manner as wind following the DOE-STD-1020 approach with importance factors of unity and increased snow load for PC-2. Note that snow load is not explicitly addressed in DOE-STD-1020 and this approach is adopted for LANL herein. The seismic importance factor from the IBC and ASCE 7, I_E is used for LANL buildings. Building categorization and snow, wind, and seismic load factors are presented in Table II-7.

Table II-7 Classification of Buildings and Importance Factors

Performance Category	PC-1	PC-2
Category (IBC Table 1604.5)	II	IV
Seismic Use Group	I	III
Seismic Design Category	D	D
Seismic Factor, I_E	1.0	1.5
Snow Factor, I_S	1.0	1.0
Wind Factor, I_W	1.0	1.0

[8] See Table 1616.3 of the IBC for the establishment of Seismic Design Categories as a function of Seismic Use Group and seismic input parameters.

F. Structural design is an iterative process that begins with a conceptual or preliminary design of the building. The building layout and materials of construction are developed to meet functional and cost requirements. For seismic design of building structures, certain lateral force resisting systems and materials of construction are not permitted. Also, a certain level of structural details is required for Seismic Design Category D, which is applicable for LANL structures. The basic seismic force resisting system and its level of detailing must be selected from Table 9.5.2.2 of ASCE 7 as one of the initial steps in the seismic design process. From the preliminary building design, the structural design is completed in the following steps:

1. Evaluate expected loads on the structure as described in Section II.1.1 of this chapter.
2. Develop a mathematical model of the structure in order to evaluate structural response to the applied loads. Modeling is described in Section II.1.4 of this chapter.
3. Perform structural response analysis to determine the load effects in structural members. Load effects are forces, stresses, and deformations resulting from the applied loads. Structural analyses are described in Section II.1.5 of this chapter. For most loads, linear elastic static response analyses are performed. Dynamic analyses and considerations of inelastic response must be addressed for seismic and blast loads.
4. Determine the maximum response of structural elements by utilizing the load combinations described in Section II.1.2 of this chapter.
5. Compare the maximum responses to acceptance criteria found in the IBC and in applicable material standards to assure that the general structural design requirements specified earlier in this section (II.1.3) are met. Assure that stresses and deformations are within acceptable limits. If stresses or deformations are unacceptable, the design must be modified and the process repeated until response stresses and deformations are acceptable. In addition, apply design measures to assure that a reliable and ductile design is achieved. Design acceptance criteria and other design measures are described in Section II.1.F of this chapter.

1.4 STRUCTURAL MODELING

1.4.1 General Requirements

A. Structural response to the loads described in Section II.1.1 shall be determined by analysis of a mathematical model of the building structure in accordance with ASCE 4 [6]. The model shall include the stiffness and strength of all elements that are significant to the distribution of forces and deformations in the structure. The model shall also represent the spatial distribution of the stiffness and mass (for seismic analyses) throughout the structure. For seismic analysis of PC-1 and PC-2 building structures, the model need only be capable of representing horizontal seismic response. For these performance categories, vertical seismic response is considered in the load combinations by a factored dead load term.

B. The model shall include all structural members of the building that experience significant effects (i.e., forces, stresses, and deformation) to the applied loads. The structural model must include the complete load path for vertical loads from the point of application to the foundation and supporting media to properly consider gravity loads. The model must also include the complete lateral load path to properly consider earthquake and wind loads.

C. The amount of detail used to represent a structure in a mathematical model depends on the structural configuration and the use of the model. Finite element mathematical models are used to represent complex structures. A detailed model of the building structure shall be prepared to evaluate the structural response to static loads such as dead load, live load, rain and snow loads, etc. This model may also be used to evaluate seismic response of the structure when the equivalent static lateral force procedure is used. When seismic dynamic analysis is required, the same detailed model may be used to evaluate seismic inertial loads and the structural response to earthquake input motion or a more simple model may be prepared which captures dynamic behavior and evaluates seismic inertial loads. For the latter case, the seismic inertial forces may then be applied to the detailed structural model to determine the seismic response in the elements of the structure.

D. The model shall accurately represent the stiffness of the structure and parts of the structure such that reasonable response deformations may be computed. Accurate representation of the stiffness also enables reasonable estimation of building dynamic characteristics for dynamic seismic analyses. For dynamic seismic analyses, the model shall accurately represent the mass distribution throughout the structure. Accurate representation of mass enables reasonable estimation of building dynamic characteristics and seismic induced inertial loads for dynamic seismic analyses. The model shall represent the actual locations of centers of masses and centers of rigidity, thus accounting for torsional effects produced by lateral loads and their eccentricity.

E. When calculating forces in various structural elements, the torsional moments due to accidental eccentricity with respect to the center of rigidity and non-vertically incident or incoherent seismic waves shall be accounted for. An acceptable means of accounting for these torsional moments is to include an additional torsional moment in the design and evaluation of structural members. This additional torsional moment shall be taken equal to the story shear at the elevation and in the direction of interest times a moment arm equal to 5% of the building plan dimension perpendicular to the direction of lateral force in the analysis. Consideration of such eccentricity shall be used only to increase the magnitude of the forces. The structural model used for seismic analysis shall not be changed to include accidental torsion.

1.4.2 Modeling of Stiffness

A. Mathematical models shall seek best estimate stiffness properties for structural elements such that reasonable deformations are computed and best estimate dynamic characteristics are achieved. The model shall comply with the following.

   1. Stiffness properties of reinforced concrete and masonry elements shall consider the effects of cracked section.
   2. For steel moment resisting frame systems, the contribution of panel zone deformations to overall story drift shall be included.

B. In general, the distribution of loads in structural elements throughout concrete and masonry structures is approximately the same whether gross “uncracked” properties or “cracked” properties are used. However, displacement will be underestimated when uncracked properties are used and there is substantial cracking in the structure. Hence, cracked section properties for concrete or masonry members in the model shall be used when such cracking is projected.

C. Cracking in reinforced concrete and masonry structures are complex phenomena generally making detailed analytical determination of the appropriate properties impractical. Good engineering judgment and experience must play a major role in considering the effect of cracking on the stiffness of concrete. When the effects of cracking are considered, section properties developed based on the maximum internal moments from the various load combinations can be estimated using Equation 9-8 in ACI 318 [16] as presented below as Equation II-4.

$$I_E = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad \text{Eq. II-4a}$$

where:

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad \text{Eq. II-4b}$$

and for normal weight concrete:

$$f_r = 7.5\sqrt{f'_c} \qquad \text{Eq. II-4c}$$

where M_a is the maximum moment in the member, M_{cr} is the cracking moment, I_g is the gross moment of inertia, I_{cr} is the moment of inertia of the cracked transformed section, f_r is modulus of rupture, and y_t is distance from centroidal axis to extreme fiber in tension.

D. When buildings are subjected to earthquakes, limited inelastic behavior is permitted. Such inelastic behavior will result in significant cracking in reinforced concrete and reinforced masonry structures. Hence for elastic seismic analysis, the effective stiffness of reinforced concrete (or reinforced masonry) members provided in Table II-8 shall be used. When finite element methods are used, the element stiffness shall be modified using the effective stiffness factor for the dominant response parameter.

E. The material properties of interest for the purposes of modeling structure stiffness are the modulus of elasticity and Poisson’s ratio. These material properties for common structural materials may be taken as follows:

Concrete:	Calculate per Section 3.1.2.1.1 of ASCE 4 [6].
Steel:	Use values given in Section 3.1.2.1.2 of ASCE 4 [6].
Masonry:	Calculate per Section 1.8.2.2.1 of ACI 530 [17].
Aluminum:	Calculate per Section 3.1.2.1.3 of ASCE 4 [6].

F. The type of finite element required to model a structural system shall depend on the type of response desired. The selection of the finite element type shall also consider the analytical theory on which the element is based, in order to represent major characteristics of the structural system. The selection of discretization parameters shall consider the size, shape, and aspect ratio of the elements; the internal node points; and the number of nodes required to define the element. The finite element model shall produce responses that are not significantly affected by further refinement in the element mesh size and shape. The latter requirement may be satisfied based on comparable past experience in lieu of multiple analyses using successively refined mesh.

Table II-8 Effective Stiffness of Reinforced Concrete Members

Member	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Beams – nonprestressed	$0.5E_cI_g$	G_cA_W	
Beams – prestressed	E_cI_g	G_cA_W	
Columns in compression	$0.7E_cI_g$	G_cA_W	E_cA_g
Columns in tension	$0.5E_cI_g$	G_cA_W	$E_s A_s$
Walls and Diaphrams – uncracked , $f_b<f_{cr}$, $V<V_c$	$0.8E_cI_g$	$0.8G_cA_W$	E_cA_g
Walls and Diaphrams – cracked, $f_b>f_{cr}$, $V>V_c$	$0.5E_cI_g$	$0.5G_cA_W$	E_cA_g
E_c = concrete compressive modulus G_c = concrete shear modulus = $0.4E_c$ E_s = steel modulus I_g = gross moment of inertia	A_w = web area A_g = gross area of the concrete section A_s = gross area of the reinforcing steel f_b = bending stress		f_{cr} = cracking stress V = wall shear V_c = nominal concrete shear capacity

1.4.3 Modeling of Mass

A. For seismic dynamic analyses, the total mass of the structure and the distribution of mass throughout the structure must be modeled. The total seismic weight of the structure shall include the dead load plus the other loads listed below:

1. A minimum of 25% of the floor live load[9].
2. Where an allowance for partition load is included in the floor design, the actual partition weight or a minimum weight of 10 psf.
3. Total operating weight of permanent equipment.

B. The inertial properties of a structure may be modeled by assuming that the structural mass and associated rotational inertia are discretized and lumped at node points. Vertical inertia effects need not be modeled for PC-1 and PC-2 structures. The following mass modeling conditions shall be met:

2. Structural mass shall be lumped so that the total mass, as well as the center of gravity, is preserved, both for the total structure and for any of its major components that respond in the direction of motion.

[9] Note that this is a conservative LANL requirement as this provision in ASCE 7 only applies in areas used for storage.

3. The number of dynamic degrees of freedom, and hence the number of lumped masses, shall be selected so that all significant vibration modes of the structure can be evaluated. For a structure with distributed mass, the number of degrees of freedom in a given direction shall be equal to at least twice the number of significant modes in that direction.

C. *The effect and location of the buildings mass can be included in one of two ways; 1) masses may be lumped at nodes by the structural analyst in a manner in which the total mass and c.g. of the structure and major components is preserved, or 2) the structural mass may be modeled using the mass density terms in the structural software used such that lumped masses are automatically generated. In the latter case, the mass from fixed equipment, partitions, and an appropriate percentage of the live load as described above must be included manually. It is generally recommended that masses be manually lumped at nodes such that the analyst best understands the structure dynamic behavior and extraneous modes are not generated that compromise numerical accuracy*.

1.4.4 Modeling of Structural Damping

A. For seismic analysis of PC-1 and PC-2 building structures, 5% of critical damping is assumed consistent with ASCE 7 and IBC seismic provisions. The ground motion parameters presented in Section II.1.1.10 correspond to 5% damping. It is anticipated that most structural analyses for PC-1 and PC-2 building structures, will employ either equivalent lateral force or modal response spectra analysis methods for which damping is included in the specified lateral forces or input response spectra. For dynamic time history analyses, 5% damping should be used for all vibration modes.

1.4.5 Soil-Structure Interaction (SSI) for PC-1 and PC-2 Building Structures

A. If the option to incorporate the effects of SSI is exercised, ASCE 7 Section 9.5.9 shall be followed.

1.5 STRUCTURAL ANALYSIS

A. Load effects consisting of member forces, stresses, and deformations shall be determined by means of structural analyses using the loads described in Section II.1.1. For all loads except for earthquake and accidental blast, structural analysis shall be accomplished by means of a linear static analysis. For earthquake or accidental blast considerations, other structural analysis methods are used as discussed in this section.

B. Seismic analyses are required to determine the effects of horizontal earthquake motion, Q_E as shown in Equation II-3. The evaluation of total earthquake effects, E, is discussed in the following section. For seismic analysis, the following analytical procedures are permitted along with the section in ASCE 7 that describes the procedure:

- Simplified Analysis – ASCE 7 Section 9.5.4
- Equivalent Lateral Force Analysis – ASCE 7 Section 9.5.5
- Modal Response Spectrum Analysis – ASCE 7 Section 9.5.6
- Linear Response History Analysis – ASCE 7 Section 9.5.7
- Nonlinear Response History Analysis – ASCE 7 Section 9.5.8

C. For PC-1 and PC-2 building structures, it is anticipated that simplified analyses, equivalent lateral force analysis, or modal response spectrum analysis will be performed. Linear response history analysis could be performed if it was also desired to also compute in-structure response spectra for evaluation of structure-supported systems and components. Nonlinear response history analysis would only be performed if isolation or energy dissipation devices are incorporated into the design. Simplified analysis, equivalent lateral force, and modal response spectrum procedures are described below. For linear or nonlinear response history analyses, the ASCE 7 sections listed above shall be followed.

D. Modal response spectrum seismic analysis is permitted for all PC-1 and PC-2 building structures. Whether simplified analysis or equivalent lateral force analysis methods may be used depends on characteristics of the building. Table 9.5.2.5.1 (ASCE 7) combined with Tables 9.5.2.3.2 and 9.5.2.3.3 defining plan and vertical structural irregularities, respectively shall be followed in order to decide what type of seismic analysis is permissible. Recall that LANL PC-1 and PC-2 structures are in Seismic Design Category D, while PC-1 structures are in Seismic Use Group I and PC-2 structures are in Seismic Use Group III, when using the above tables.

E. Structural analysis for accidental blast shall be conducted using nonlinear response history analysis. Such analyses for blast evaluation are also briefly described below.

1.5.1 Simplified and Equivalent Lateral Force Seismic Analysis Methodology

A. Seismic analysis by the simplified analysis procedure (Section 9.5.4 of ASCE 7) or the equivalent lateral force analysis procedure (Section 9.5.5 of ASCE 7) are relatively simple static analysis methods where static lateral loads are applied to simulate earthquake ground motion input to the structure. The simplified analysis approach uses the peak ground response spectra acceleration and, thus does not require knowledge of the dynamic characteristics of the structure. The equivalent lateral force approach establishes the static lateral loads representing the earthquake input as a function of the building/structure natural period such that an estimate of the structure dynamic characteristics is needed. However, only the fundamental period is needed and equations are provided to estimate this value.

1.5.2 Simplified Analysis Procedure

A. The simplified analysis approach is only permitted for PC-1 building structures not exceeding two stories in height. The seismic base shear in a given direction shall be determined as follows:

$$V = \frac{1.2S_{DS}}{R} W = \frac{0.65W}{R} \qquad \text{Eq. II-5}$$

where S_{DS} is 0.54g as specified in Section II.1.1, W is the effective seismic weight of the structure, and R is a response modification coefficient. R values are given in Table 9.5.2.2 of ASCE 7 as a function of the characteristics of the basic seismic force-resisting system, including structural system, materials of construction, and level of detailing. R reduces earthquake loads as a means of accounting for limited inelastic behavior during earthquake ground shaking. ASCE 7 rules in Section 9.5.2 shall be followed for selecting the appropriate value of R in each direction considered.

B. The forces at each level of the structure shall be calculated as follows:

$$F_x = \frac{1.2S_{DS}}{R} w_x = \frac{0.65w_x}{R} \quad \text{Eq. II-6}$$

where w_x is the portion of the effective seismic weight of the structure, W, at level x. Horizontal distribution of forces shall be by tributary area for flexible diaphragms or by relative rigidity of vertical systems for rigid diaphragms. Diaphragms constructed of wood structural panels or untopped steel decking are permitted to be considered as flexible.

1.5.3 Equivalent Lateral Force Procedure

A. The equivalent lateral force approach is permitted for PC-1 and PC-2 building structures as shown below:

- Regular structures with fundamental period less than 1.7 seconds and all light-frame construction
- Irregular structures with fundamental period less than 1.7 seconds having only plan irregularities of Type 2, 3, 4, or 5 of ASCE 7 Table 9.5.2.3.2 or vertical irregularities of Type 4 or 5 of ASCE 7 Table 9.5.2.3.3

B. This approach is not permitted for all other structures.

1. The seismic base shear in a given direction for the equivalent lateral force procedure shall be determined as follows:

$$V = C_s W = \frac{S_{DS}}{R/I_E} W = \frac{0.54W}{R/I_E} \quad \text{Eq. II-7a}$$

2. The seismic base shear need not be greater than:

$$V = C_s W = \frac{S_{D1}}{T(R/I_E)} W = \frac{0.26W}{T(R/I_E)} \quad \text{Eq. II-7b}$$

3. The seismic base shear shall not be taken less than:

$$V = C_s W = 0.044 S_{DS} I_E W = 0.024 I_E W \quad \text{Eq. II-7c}$$

where S_{DS} is 0.54g and S_{D1} is 0.26g as specified in Section II.1.A, W is the effective seismic weight of the structure, R is a response modification coefficient, I_E is the importance factor, and T is the fundamental period of the structure in the direction being considered. I_E is 1.0 for PC-1 structures and 1.5 for PC-2 structures as presented in Section II.1.C. The above expression intentionally retains the term R/I_E rather than increasing the value in the numerator for PC-2 structures. The philosophy for the importance factor is not to increase the earthquake load but rather to decrease the response modification coefficient such that less inelastic behavior is permitted and the ability of the structure to remain in operation is enhanced.

4. Minimum levels of seismic base shear for LANL building structures shall be:

$$V_{min} = 0.044 S_{DS} I W$$
$$V_{min}(PC-1) = 0.024W \qquad \text{Eq. II-8}$$
$$V_{min}(PC-2) = 0.036W$$

C. The fundamental period of the structure in each horizontal direction may be determined by analysis or by the approximate equations given in Section 9.5.5.3 of ASCE 7. In either case, the period T to be used in the evaluation of the base shear by Equation II-7b, shall be no greater that the coefficient, C_u times the approximate fundamental period, T_a. C_u is a function of input parameter, S_{D1} and for LANL structures, C_u shall be taken as 1.44. Hence, the maximum value of T shall be;

$$T_{max} = 1.44 T_a \qquad \text{Eq. II-9}$$

where T_a is determined by the equations and tables in ASCE 7 section 9.5.5.3.2.

D. The base shear, V, as determined above shall be distributed vertically and horizontally to members of the lateral force resisting systems of the building structure in accordance with ASCE 7 Sections 9.5.5.4, 9.5.5.5 and 9.5.5.6.

E. The load effects including forces, stresses, and deformation throughout the structure are determined from the linear static analysis for the earthquake loads as defined above. The resulting displacements must be used to evaluate story drift and to assess potential P-Delta effects. The design story drift, Δ is computed as the difference of the deflections at the top and bottom of the story under consideration. Story drift is determined from the following:

$$\Delta = \delta_x - \delta_{x-1}$$
$$\delta_x = \frac{C_d \delta_{xe}}{I_E} \qquad \text{Eq. II-10}$$

where δ_x is the deflection of Level x in the structure and δ_{xe} is the deflection at Level x determined by the elastic analysis subjected to earthquake load in accordance with Equation II-7 and C_d is the deflection amplification factor. C_d values are given in Table 9.5.2.2 of ASCE 7 as a function of the characteristics of the basic seismic force-resisting system, including structural system, materials of construction, and level of detailing. Note that the upper limit of fundamental period given in Equation II-9 need not apply for the evaluation of drift only.

F. Where P-Delta effects are found to be significant, the resulting earthquake load effects (forces, stresses, and deformations) must be amplified. The significance of P-Delta effects are determined by the evaluation of a stability coefficient, θ, as follows:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \qquad \text{Eq. II-11}$$

where P_x is the total vertical design load at and above Level x (when computing P_x, no individual load factor need exceed 1.0). V_x is seismic shear force acting between Levels x and x-1, and h_{sx} is the story height below Level x. Δ is the story drift associated with shear force V_x and C_d was defined previously.

G. P-Delta effects are not considered significant when θ is equal to or less than 0.10. The structure is considered unstable if θ is greater than θ_{max} as defined below:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25 \qquad \text{Eq. II-12}$$

where β is the ratio of shear demand to shear capacity for the story between Level x and x-1. This ratio may be conservatively taken as 1.0. When θ is greater than θ_{max}, the structures is potentially unstable and shall be redesigned. When θ is between 0.10 and θ_{max}, the story drift and member forces and moments shall be amplified by the ratio of 1.0/(1-θ).

1.5.4 Modal Response Spectra Analysis Methodology

A. For highly irregular PC-1 and PC-2 structures, as defined in Table 9.5.2.5.1 of ASCE 7, modal response spectra analysis shall be used to evaluate the effects of horizontal earthquake ground motion, Q_E. This method may be used for any PC-1 or PC-2 structure. Required standards for this approach are given in Section 9.5.6 of ASCE 7 and briefly summarized in this section. The symbols used in this method of analysis have the same meaning as those for similar terms in the equivalent lateral force procedure, with the subscript m denoting quantities in the m^{th} mode.

B. An analysis shall be conducted to determine the natural modes of vibration for the structure, including the period for each mode, the modal shape vector, Φ, the modal participation factor, and modal mass. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90% of the actual mass in each of two orthogonal horizontal directions. The required periods, mode shapes, and participation factors of the structure in the direction under consideration shall be calculated by established methods of structural analysis for the fixed base condition using the masses and elastic stiffness of the seismic force-resisting system.

C. The portion of the base shear contributed by the m^th^ mode, V_m, shall be determined from the following equations:

$$V_m = C_{sm} W_m$$

$$W_m = \frac{\left(\sum_{i=1}^{n} w_i \phi_{im}\right)^2}{\sum_{i=1}^{n} w_i \phi_{im}^2} \qquad \text{Eq. II-13}$$

where C_{sm} is the modal seismic design coefficient defined below, W_m is the effective modal gravity load, w_i is the portion of the total gravity load of the structure at Level i, and ϕ_{im} is the mode shape displacement at the i^{th} level of the structure when vibrating in the m^{th} mode. The modal seismic design coefficient shall be determined in accordance with the following:

$$C_{sm} = \frac{S_{am}}{R/I_E} \qquad \text{Eq. II-14}$$

where S_{am} is the design spectral response acceleration at period T_m from Figure II-1. Exceptions to Equation II-14 are:

$$C_{sm} = \frac{S_{DS}}{R/I_E} = \frac{0.54}{R/I_E} \text{ for } T_M \leq 0.096 \text{ seconds}$$

$$C_{sm} = \frac{4S_{D1}}{(R/I_E)T_m^2} = \frac{1.04}{(R/I_E)T_m^2} \text{ for } T_M \geq 4.0 \text{ seconds} \qquad \text{Eq. II-15}$$

H. Modal deflection at each Level x, δ_{xm}, shall be determined by:

$$\delta_{xm} = \frac{C_d \delta_{xem}}{I_E} \qquad \text{Eq. II-16}$$

where δ_{xem} is the deflection of Level x in the m^{th} mode at the center of mass at Level x determined by an elastic analysis.

I. The story shears, story overturning moments, and the shear forces and overturning moments in vertical elements of the structural system at each level due to the seismic forces determined from the modal seismic forces defined by ASCE 7 Section 9.5.6.6 shall be computed for each mode by linear static methods. Alternately, if a sufficiently detailed model is used for the dynamic analysis, these quantities may be determined directly by that analysis. These modal quantities shall be combined by the modal combination methods as described in ASCE 4 Section 3.2.7.1 to obtain design values for these parameters (story shears, etc.).

J. A base shear, V, shall be calculated using the equivalent lateral force procedure as given by Equation II-7. This base shear will be compared to the base shear resulting from the combined modes from the modal analysis, V_t. Where V_t is less than 85% of V, the design story shears, moments, drifts, and floor deflections shall be multiplied by the factor, $0.85V/V_t$.

K. P-Delta effects shall be considered in the same manner as described for the equivalent lateral force procedure. When θ is between 0.10 and θ_{max}, the story drift and member forces and moments shall be amplified by the ratio of $1.0/(1-\theta)$.

1.5.5 Response History Analysis Methodology

A. Linear and nonlinear response history analyses for seismic evaluation are described in ASCE 7 Sections 9.5.7 and 9.5.8, respectively. As mentioned previously, these approaches are expected to be used very infrequently for PC-1 and PC-2 structures at LANL. Hence, there is no further discussion of these methods in this chapter.

1.5.6 Evaluation of Blast Effects

A. PC-1 and PC-2 structures may be subjected to blast effects from either designed experiment blast loads, L_{EB}, or from accidental blast loads, A_B. For designed experiment blast conditions, it is required that shielding or containment be provided by internal structures such that blast loads on the PC-1 and PC-2 building structure are minimized. As mentioned previously, the design of internal containment structures or shielding for experimental explosions is not within the scope of this chapter. However, the building could experience reaction forces from the internal containment structure or blast effects from exterior experimental explosions. Since the building structure may be subjected to these experimental blast loads repetitively, the resulting loads must be sufficiently small that progressive damage to the building does not occur. Structural response must be limited to elastic behavior. Hence, the designed experiment blast loads, L_{EB}, on the building structure are consistent with live load. Experimental blast loads are short duration, pulse loads. *Analysis for such loads may be conservatively performed by linear static analysis at the peak loading. Alternately, the dynamic nature of the load may be accounted for to obtain more realistic results.*

B. This section covers structural analysis of the building structure for accidental blast, A_B loadings. Accidental explosions may occur during handling of high explosive materials resulting in a detonation or due to a release of hydrocarbons followed by ignition resulting in a vapor cloud explosion or deflagration. In either case, the resulting loads on building structures can be of very large amplitude depending on the distance of the building from the explosion, but these loads will be of very short duration in a single pulse, on the order of a fraction of a second.

C. Because of the short duration characteristic of blast loadings, the dynamic characteristics of the building structure typically effectively reduces the blast effects, as the period of the structure is usually much greater than the duration of the load. In addition, ductile structures can undergo extensive inelastic behavior, short of collapse, during these short duration blast loads. The short duration blast pulse imparts relatively low energy to the structure that is absorbed during strain energy of structural members responding in the nonlinear range.

D. Because the amplitude of blast overpressure acting on building surfaces can be very large compared to earthquake or wind forces acting on these surfaces, it is necessary to account for yielding of building structural members in order to obtain an economical design. For this reason and to account for dynamic effects, structural analysis for accidental blast loads is accomplished by nonlinear response history analyses. Such analysis may be accomplished by nonlinear, dynamic finite element computer programs. Alternately, there are more simple approximate methods that fully account for both the dynamic character of the structure and the blast load; and the nonlinear behavior of the structure withstanding the blast loads.

E. The first step in the blast structural evaluation is to evaluate the loads acting on the building walls and roof in terms of blast overpressure, impulse, and duration at the building location. Building walls facing the explosion source will be subjected to increased reflected pressure. Hence, the location of potential explosion sources must be identified and accounted for in the determination of building loads. The walls not facing the explosion source and the roof will also be subjected to blast pressure loads. Techniques are available for estimating building loads in TM 5-1300 [22], DOE/TIC-11268 [32], and the ASCE report on blast resistant design in petrochemical facilities [44].

F. Building or component response is then determined by nonlinear response time history analyses or by simplified approximate methods per TM 5-1300 [22] or other similar references. Techniques for these analyses are found in the references cited above. Response quantities of interest include forces, moments, stresses, and displacements; as well as plastic hinge locations and support rotations and ductility levels to measure the level of response inelastic behavior.

1.6 Acceptance Criteria and Other Design Requirements

A. The basic structural design requirements are presented in Section II.1.3. This section provides the measures that demonstrate that those requirements are met. For non-seismic or non-blast loads, the members of the structure must have sufficient strength and stiffness to withstand the applied loads in accordance with ASCE 7, the IBC, or the applicable material standard. For seismic and blast loads, the structure must also have sufficient strength and stiffness to withstand the applied loads. However, for these transient, limited energy, dynamic loads, the structure must also have sufficient energy dissipation capacity. This latter requirement necessitates a number of required design detailing measures that are necessary to achieve satisfactory structural performance.

B. The basic provisions for assuring structural members have adequate strength and stiffness to withstand all applied loads and the additional design requirements essential to achieve satisfactory seismic or blast structural performance are presented in this section.

C. For structural design for accidental blast loads, acceptance criteria is typically stated in terms of support rotations and ductility ratios by component/construction type for various levels of protection. The amount of acceptable damage is the selected by the designer. *Because of the infrequent nature of accidental explosions, high levels of damage corresponding to structure replacement after the event is reasonable.* For blast design considerations, structural capacities shall be evaluated in accordance with strength or plastic design principles (ACI-318 or AISC LRFD) without capacity reduction factors, ϕ, and with dynamic increase factors per the guidance in References 22, 32, or 44.

1.6.1 Evaluation of Total Demand

A. Following the structural analysis methods described in Section II.1.5, the element forces, moments, and/or stresses, and joint displacements are determined for each of the loads described in Section II.1.1. From the seismic analysis, the effects of horizontal seismic (earthquake-induced) forces, Q_E is determined. Q_E includes the effects of inelastic energy absorption of the structure during transient earthquake ground motion as the elastic response has been reduced by the factor R/I_E. The effect of horizontal and vertical earthquake-induced forces, E, to be used in load combinations and to compare to acceptable limits must be evaluated using Equation II-3a. Use of this equation requires the evaluation of the reliability coefficient, ρ, based on the extent of structural redundancy present in a building as defined in ASCE 7 Section 9.5.2.7 and the overstrength factor, Ω_o, as defined in ASCE 7 Table 9.5.2.2 as a function of the characteristics of the basic seismic force-resisting system, including structural system, materials of construction, and level of detailing.

B. The next step is to combine responses from seismic and other concurrent loadings to evaluate the total demand forces, D_{TI} for the applicable load combinations described in II.1. 2. *For building structures, a computer analysis will normally be performed. It is advantageous to apply each non-seismic load case and the seismic load cases one at a time. Through diligent use of the load combination capabilities of the computer software, the load combinations in II.1.2 can be readily performed and the possibility of a member response being governed when one load is not active (when it counteracts the other loads) may be systematically investigated.*

1.6.2 Reinforced Concrete Structure Design Requirements

A. PC-1 and PC-2 reinforced concrete structures shall be designed and detailed in accordance with ACI 318 as referenced below and Appendix A9.9 of ASCE 7:

- American Concrete Institute (ACI), “Building Code Requirements for Structural Concrete,” ACI 318, excluding Appendix A.

B. The provisions of ACI 318 [16] shall be applied for determining the construction requirements and capacity of the reinforced concrete components of PC-1 and PC-2 structures. The basic requirement of ACI 318 is that the capacity, which is equal to ΦM_n, ΦP_n, and ΦV_n (where M_n, P_n, and V_n are the nominal capacities using ultimate strength methods and Φ are the appropriate strength reduction factors) is greater than the limiting demand (M_u, P_u, and V_u) calculated using the load combinations in Section II.1.2.

C. Special provisions for Seismic Category D LANL PC-1 and PC-2 reinforced concrete structures include:

- Concrete moment frames used to resist seismic forces shall be special moment frames.
- Shear walls used to resist seismic forces shall be special reinforced concrete shear walls.
- Frame components assumed not to contribute to lateral force resistance shall conform to Section 21.11 of ACI 318 such that they can resist gravity loads at lateral displacements due to earthquake response.
- Columns supporting reactions from discontinuous stiff members such as walls shall be designed for the special seismic load combination (Equation II-3b) and shall have transverse reinforcement in accordance with ASCE 7 Section A.9.9.4.2.

D. Section 10.11, 10.12 and 10.13 of ACI 318 include a methodology for determining whether moments on column members should be modified due to secondary effects (P-Δ). The provisions of these sections may be applied for concrete structures in place of the methodology for considering (or not considering) P-Δ effects discussed in Section II.1.5. In no case should both be applied, as that would double count the additional moment on the building structure due to secondary effects.

1.6.3 Steel Structure Design Requirements

A. PC-1 and PC-2 steel structures shall be designed and detailed in accordance with the latest version of the documents referenced below, other documents referenced in Section 9.8 of ASCE 7, and Appendix A9.8 of ASCE 7:

- American Institute of Steel Construction (AISC), "Load and Resistance Factor Design Specification for Structural Steel (LRFD)."
- American Institute of Steel Construction (AISC), "Seismic Provisions for Structural Steel Buildings," ANSI/AISC 341.
- American Iron and Steel Institute (AISI), "Specification for the Design of Cold-Formed Steel Structural Members," 1996, including Supplement No. 1.

B. The basic requirement for LRFD is that the capacity, which is equal to ΦM_n, ΦP_n, and ΦV_n (where M_n, P_n, and V_n are the nominal capacities using ultimate strength methods and Φ are the appropriate strength reduction factors) is greater than the limiting demand (M_u, P_u, and V_u) calculated using the load combinations in Table II-5.

C. Special provisions for Seismic Category D LANL PC-1 and PC-2 steel structures include:

- Steel structures shall be designed and detailed in accordance with LRFD provisions and Part I of the AISC seismic provisions.
- Light-framed cold-formed steel wall systems shall be designed and detailed in accordance with Section A.9.8.6 of ASCE 7.
- For cold-formed steel structures shall be designed for seismic loads in accordance with the AISI specification listed above with the exception that Section A5.1.3 of that specification is revised by deleting reference to earthquake or seismic loads in the sentence permitting the 0.75 factor. The load combinations in this Chapter (Table II-5) shall be used.
- Steel deck diaphragms shall be made of materials conforming to the requirements of the AISI specification listed above or ASCE 8 [53]. Nominal strengths shall be determined in accordance with approved analytical procedures or with test procedures prepared by a registered design professional experienced in testing of cold-formed steel assemblies. Nominal strengths shall be approved by the LANL Chapter 5 POC. Design strengths shall be determined by multiplying the nominal strength by a resistance factor, ϕ equal to 0.60 for mechanically connected diaphragms and equal to 0.50 for welded diaphragms. The steel deck installation for the building, including fasteners, shall comply with the test assembly arrangement. Quality standards established for the nominal strength test shall be the minimum standards required for the steel deck installation, including fasteners.
- The design strength of steel cables shall be determined by the provisions of ASCE 19-96 [54], with the following exceptions. Section 5d of the standard shall be modified by substituting $1.4(T_4)$ when T_4 is the net tension in the cable due to dead load, prestress, live load, and seismic load. A load factor of 1.1 shall be applied to the prestress force to be added to the load combination of Section 3.1.2 of the standard.

- Chapter C of AISC-LRFD [13] includes a methodology for determining whether moments on column members should be modified due to secondary effects (P-Δ). The provisions of this section may be applied for steel structures designed using LRFD in place of the methodology for considering P-Δ effects discussed in Section II.1.E.
- Where the AISC seismic provisions require Amplified Seismic Loads, Equation II-3b shall be used to establish seismic loads.

1.6.4 Reinforced Masonry Structure Design Requirements

A. PC-1 and PC-2 reinforced masonry structures shall be designed and detailed in accordance with ACI 530 as referenced below and Appendix A9.11 of ASCE 7:

- American Concrete Institute (ACI), "Building Code Requirements for Masonry Structures," ACI 530/ASCE 5/TMS 402 and "Specifications for Masonry Structures," ACI 530.1/ASCE 6/TMS 602.

B. References to "Seismic Performance Category" in Section 1.13, and elsewhere in ACI 530, shall be replaced by "Seismic Design Category." Other required exceptions to ACI 530 are presented in ASCE 7 Appendix A9.11.

1.6.5 Aluminum Structure Design Requirements

A. PC-1 and PC-2 aluminum structures shall comply with the provisions of the Aluminum Design Manual (ADM) [36]. When applying the ADM, the nominal loads shall be the minimum design loads required by Section II.1.1.

B. Part 1-A of the ADM, Specifications for Aluminum Structures – Allowable Stress Design shall be applied in its entirety except the load combinations defined in Table II-6 shall be followed. The one-third increase of the allowable stresses provided in the ADM, Part 1-A, Section 2.3, for wind or seismic loading, acting alone or in combination with dead load or live load shall not be used.

C. Part 1-B of the ADM, Specifications for Aluminum Structures – Load and Resistance Factor Design of Buildings and Similar Type Structures shall be applied in its entirety except the load combinations defined in Table II-5 shall be followed.

D. Testing shall be considered to be an acceptable method for substantiating the design of aluminum alloy load carrying members, assemblies or connections whose strengths cannot otherwise be determined. The testing shall follow the guide of Section 8 of either Part I-A or Part I-B of the aluminum code [36].

1.6.6 Drift Limits

A. The story drift due to seismic response, Δ, as determined by Equation II-10 or an equivalent modal analysis relation shall not exceed the allowable story drift for any story as shown in Table II-9. For structures with significant torsional deflections, the maximum drift shall include torsional effects. All portions of the structure shall be designed and constructed to act as an integral unit in resisting seismic forces unless separated structurally by a distance sufficient to avoid damaging contact in accordance with Section II.1.6.7.

Table II-9 Allowable Story Drift, Δ_a

Structure	PC-1	PC-2
Structures, other than masonry shear wall or masonry wall frame structures, four stories or less with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025h_{sx}$[10]	$0.015h_{sx}$
Masonry cantilever shear wall structures[11]	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$
Masonry wall frame structures	$0.013h_{sx}$	$0.010h_{sx}$
Other structures	$0.020h_{sx}$	$0.010h_{sx}$

B. When the simplified analysis procedure for seismic design of buildings is employed, the design story drift, Δ, shall be taken as 1% of the story height unless a more exact analysis is provided.

1.6.7 Building Separation

A. Consider the relative head-on deformations between adjacent structures to prevent potential pounding during lateral loads due to earthquake or wind. A minimum separation of S, calculated as follows, shall be maintained between adjacent structures:

$$S = 2.0\,(\delta_{x1}^2 + \delta_{x2}^2)1/2 \qquad \text{Eq. II-17}$$

where δ_{x1} and δ_{x2} are the maximum displacements along the same axis for the adjacent building structures computed in accordance with Equations II-10 or II-16.

1.6.8 Foundations

A. Permanent buildings[12] and similar structures shall have a permanent foundation meeting the design requirements in Chapter 18 of the IBC [5]. Foundations shall provide full perimeter support and protection against rodent infestation. *LANL experience is that permanent foundations reduce operations and maintenance costs by minimizing settling that causes roof and structure cracks, excluding rodents and other pests, and improving energy efficiency by virtue of superior insulation characteristics.*

[10] h_{sx} is the story height below Level x

[11] Masonry cantilever shear wall structures are structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

[12] Permanent is defined in ESM Chapter 1, Section Z10.

1.7 ADDITIONAL STRUCTURAL DESIGN CONSIDERATIONS

1.7.1 Minimum Antiterrorism Structural Design Measures

A. Structural design measures on progressive collapse avoidance and on window protection presented in the DoD Minimum Antiterrorism Standards for Buildings [46] shall be considered for those buildings where there is a credible terrorist threat at the building. LANL shall specify whether these minimum antiterrorism measures are to be implemented. The following guidelines are provided to aid the project manager in making this determination[13]:

- ML-1 or ML-2 nuclear facilities
- Buildings with high occupancy (greater than 300 occupants)
- Buildings with high consequence of failure (high risk, essential mission, etc.)
- Important buildings for which potential terrorist threats have not been mitigated by security measures

B. Progressive collapse provisions from Ref. 46 are additions to the continuity and redundancy requirements for seismic design. By these provisions, columns are designed to accommodate the loss of lateral support from one floor such that columns are designed for increased unbraced length. In addition, the structure is designed such that an exterior member (beam or column) can be removed without collapse of the building. Also, all floors are to be designed with improved capacity to withstand load reversals due to explosive effects by designing them to withstand a net uplift equal to the dead load plus one half the live load. Provisions for windows include the use of laminated glass of a required minimum thickness and the design of window frames for a minimum load on the glass surface.

1.7.2 Seismic Design and Detailing Requirements

A. The design and detailing of the components of the seismic force resisting system shall comply with Section 9.5 of ASCE 7, the IBC, and applicable material standards. This section highlights some important design and detailing requirements from these other sources.

B. All parts of the structure between separation joints shall be interconnected to form a continuous path to the seismic force resisting system, and the connections shall be capable of transmitting the seismic force, F_p induced by the parts being connected. Any smaller portion of the structure shall be tied to the remainder of the structure with elements designed for a seismic force equal to $0.133S_{DS}$ times the weight of the smaller portion.

C. Where openings occur in shear walls, diaphragms, or other plate-type elements, reinforcement at the edges of the opening shall be designed to transfer the stresses into the plate structure. The edge reinforcement shall extend into the body of the wall or diaphragm a distance sufficient to develop the force in the reinforcement. The extension must be sufficient in length to allow dissipation or transfer of the force without exceeding the shear and tension capacity of the diaphragm or the wall.

D. Redundancy shall be considered in structural design. The design of a structure shall consider the potentially adverse effect that failure of a single member, connection, or component of the seismic force resisting system will have on the stability of the structure.

[13] This list is not meant of be all inclusive, but is provided to assist the project manager in specifying minimum requirements for antiterrorism.

E. Collector elements (e.g., drag struts) shall be provided that are capable of transferring the seismic forces originating in portions of the structure to the element providing the resistance to those forces. Collector elements, splices, and their connections to resisting elements shall resist the special seismic loads of Equation II-3b. Note that the quantity $\Omega_o Q_E$ in Eq. II-3b need not exceed the maximum force that can be transferred to the collector by the diaphragm and other elements of the lateral force-resisting system.

F. Concrete and masonry walls shall be anchored to the roof and all floors and members that provide lateral support for the wall or which are supported by the wall. The anchorage shall provide a direct connection between the walls and the roof or floor construction. Anchorage of walls to flexible diaphragms shall have the strength to develop the out-of-plane force given by Eq. 9.5.2.6.3.2 of ASCE 7. In accordance with ASCE 7 Section 9.5.2.6.3.2, diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorage forces into the diaphragms. The continuous ties required shall be in addition to diaphragm sheathing for wood diaphragms and the metal deck shall not be used as the continuous ties for metal deck diaphragms. The strength design forces of steel elements of the wall anchorage system, other than anchor bolts and reinforcing steel shall be 1.4 times the forces otherwise required by Section 9.5.2.6.3.2 of ASCE 7. Diaphragm to wall anchorage using embedded straps shall be attached to, or hooked around, the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

G. Columns, beams, trusses, or slabs supporting discontinuous walls or frames of structures having out-of-plane offset plan irregularities or the vertical irregularity of in-plane discontinuity in vertical lateral force resisting elements shall have the design strength to resist the maximum axial force that can develop in accordance with the special seismic loads of Equation II-3b.

H. The vertical component of earthquake ground motion shall be considered in the design of horizontal cantilever and horizontal prestressed components. The load combinations used in evaluating such components shall include E as defined by Equation II-3a. Horizontal cantilever structural components shall be designed for a minimum net upward force of 0.2 times the dead load in addition to the applicable load combinations.

I. The deflection in the plane of a diaphragm shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached elements to maintain structural integrity and continue to support gravity loads.

1.7.3 Additional Foundation Detailing Requirements

A. Permanent buildings and similar structures shall have a permanent foundation meeting the IBC (e.g., full perimeter support, rodent-excluding, no trailer skirting). *Permanent is defined in ESM Chapter 1 Section Z10.*[14]

[14] LANL experience is that permanent foundations reduce O&M costs by minimizing settling that causes roof and structure cracks, excluding rodents and other pests, and improving energy efficiency by virtue of their superior insulation

B. In addition to detailing requirements given in ASCE 7 Section 9.7 and the IBC, detailing of PC-1 and PC-2 building foundations shall meet the following requirements.[15]:

- Interconnect all spread footing type foundations using tie beams. The tie beam shall be capable of resisting, in tension or compression, a minimum horizontal force equal to 10% of the larger column vertical load. The tie beams shall also be capable of resisting bending due to prescribed differential settlements of the interconnected footings, as stipulated by the project geotechnical engineer.
- Foundations shall not be designed in locations that are within 50 feet of known active faults.
- To mitigate potential differential movements associated with surface faulting during the design level earthquake, the design shall provide for a minimum horizontal and vertical differential movement between footings of 1/2 inch for PC-2 structures unless greater movement is indicated by the geotechnical investigation.
- The perimeter basement walls and other subterranean structural walls shall be designed for soil pressures, including potential seismic loads, (Subsection A.8) as recommended by a licensed geotechnical engineer knowledgeable of LANL soil conditions.

1.7.4 Preferred Structural Properties of Buildings

A. The R factors given in ASCE 7 imply that inelastic deformation is permissible when structures are subjected to earthquake loadings. *Inelastic deformation means damage (although limited) to the structure and, after the design basis event, the structure may need to be repaired before it can be placed in full service. Certain principles should be adhered to in detailing new buildings to assure acceptable behavior during earthquakes. Suggested guidance includes:*

- *Minimize Structural Irregularities: Structure layout and detailing should minimize both horizontal and vertical irregularities.*
- *Provide a uniform strength distribution: Members of the lateral load resisting system should be designed with nearly uniform design margin for lateral loads, in order to create a system where inelastic deformations are well-distributed throughout the structure*
- *Maximize structural redundancy: Structures with large redundancy behave better under earthquake loads. The layout of the structure should include as much redundancy as practical.*

1.7.5 Specification Templates (Programmatic and Facility)

A. The LANL Standards Program maintains standard specifications for vendors that provide and construct PC-1 and PC-2 building structures as well as some nonstructural components at the LANL site. *These following specifications may be edited to suit the particular project.*

1. Section 03100 – Concrete Formwork
2. Section 03200 – Concrete Reinforcement
3. Section 03300 – Reinforced Concrete
4. Section 04300 – Unit Masonry System
5. Section 05120 – Structural Steel

[15] These provisions are in addition to the detailing requirements in IBC 2003 and are considered to be additional LANL specific requirements that are conservatively applied.

6. Section 05210 – Steel Joists and Joist Girders
7. Section 05311 – Steel Roof Deck
8. Section 05313 – Steel Floor Deck
9. Section 13125 – Pre-Engineered Metal Buildings
10. Section 05520 – Handrails and Railings
11. Section 05531 – Gratings and Floor Plates
12. Section 05400 – Cold Formed Metal Framing
13. Section 13074 – Pressure Relief Wall Panels
14. Section 13085 – Seismic Protection
15. Section 14645 – Bridge Cranes

2.0 DESIGN AND ANALYSIS REQUIREMENTS FOR PC-1 & PC-2 NON-STRUCTURAL SYSTEMS &COMPONENTS & NON-BUILDING STRUCTURES (PROGRAMMATIC & FACILITY)

A. **Nonstructural systems and components** include architectural, mechanical, and electrical components and systems located within buildings. These systems and components are supported from the floors, walls, or roof of the building. In addition, some of these systems and components may interconnect facilities. Examples include supports for piping running between facilities and supports for cable trays in which power cables run between facilities. For purposes of this ESM chapter (structural), nonstructural components are characterized primarily by the fact that they add mass to a structure but do not contribute to the strength of the lateral and vertical load resisting systems of the primary structure. Examples of nonstructural components and systems include:

- Mechanical and electrical equipment,
- Architectural components,
- Glove boxes,
- Storage racks
- Platforms and walkways,
- Cranes and hoists,
- Storage tanks and pressure vessels supported within a building
- Cable trays, conduit, and piping.

B. **Non-building structures** include all self-supporting structures other than buildings. These structures are located outside buildings on their own foundation. Examples of non-building structures include:

- Storage tanks
- Stacks and chimneys
- Storage racks
- Gas transmission and distribution piping systems
- Pressure vessels
- Impoundment dikes and walls

C. This chapter provides design and seismic qualification requirements for non-building structures, nonstructural components, equipment, and distribution systems. Specific design requirements for many of these systems are primarily included in Chapter 6 – “Mechanical” and Chapter 7- “Electrical” of this ESM. In addition, specific design requirements for these items may be in accordance with approved industry standards explicitly for the items considered.

D. For seismic design of nonstructural components, equipment, and distribution systems, there are several unique characteristics that must be considered, which are distinctly different from building seismic design. These characteristics are:

1. These systems and components may be supported by a building structure such that the input earthquake shaking is the building response to the earthquake rather than the earthquake ground shaking.

2. These systems and components may be supported at multiple points at various locations within a building or at the ground and in the building. In this case, each support point will experience a different level of shaking imposing differential deformations to the systems and components in addition to shaking response.
4. Nonstructural components, equipment, and distribution systems are typically much simpler than building structures such that they are likely to have less redundancy and inelastic energy absorption capacity.

E. Item 3 above also applies to non-building structures. These characteristics are accounted for in the design and analysis requirements of PC-1 and PC-2 nonstructural components, equipment, and distribution systems and non-building structures presented in the remainder of this chapter.

2.1 LOADS AND LOAD COMBINATIONS

2.1.1 Loads on Nonstructural Components and Non-Building Structures

A. Structural loads for PC-1 and PC-2 buildings have been discussed in Section II.1.1. These same loads generally apply to systems and components and non-building structures. For the purpose of designing supports and anchorage, the primary loads on nonstructural components, equipment, and distribution systems are dead load and seismic loads. These systems and components typically will not be subjected to lateral soil pressure loads and designed experiment blast loads. Systems and components located within buildings will not be subjected to wind, rain, snow, or flood loads. Non-building structures located out in the open will be subjected to wind, rain, snow, or flood loads.

B. Platforms and walkways as well as cable trays may have significant live loads. In addition, cranes and hoists as well as some mechanical equipment have significant operational loads. Operational loads are included in load combinations as live loads. Systems and components can also be subjected to self-straining loads and accidental blast loads in certain situations. Storage tanks are subject to fluid loads.

C. Non-building structures shall be designed for wind loads in accordance with Section 6.5.13 of ASCE 7 [11]. This approach utilizes the velocity pressure as determined in accordance with Equation II-1 combined with net force coefficients given in Figures 6-18 through 6-22 of ASCE 7.

D. Self straining forces for nonstructural components are typically thermal loads. Thermal loads are considered self limiting, however, these loads shall be considered as primary loads when performing evaluation of nonstructural components and distribution systems. *Thermal loads are primarily a local effect on piping system supports, where long continuous piping segments are subject to thermal expansion that is restricted by the supports.* Piping supports shall be designed for the forces induced from the ambient temperature of the piping *(normally considered the average environmental temperature or 70^0 F)* to the maximum normal operating temperature of the piping system.

E. Supports and anchorage for nonstructural components shall be designed for earthquake loads in accordance with Section 9.6 of ASCE 7 [11]. Non-building structures shall be designed for earthquake loads in accordance with Section 9.14 of ASCE 7 [11]. In either case, the earthquake loads for PC-1 and PC-2 are given in terms of S_{DS} (5% damped design spectral response acceleration at short periods [0.2 sec.]) and S_{D1} (5% damped design spectral response acceleration at one second period). These seismic design parameters have been developed for LANL in Section II.1.1.10 of this chapter as:

- $S_{DS} = 0.54g$
- $S_{D1} = 0.26g$

F. These parameters are used to develop equivalent static seismic loads for equivalent static force analysis methods. Alternately, modal analysis methods may be used for nonstructural components and non-building structures using earthquake input motion defined in terms of a 5% damped ground response spectrum developed from S_{DS} and S_{D1} as given in Figure II-1 for LANL PC-1 and PC-2 building structures, nonstructural components, and non-building structures.

G. Nonstructural components and non-building structures could be subjected to accidental blast loads due to either inadvertent detonation of high explosives or vapor cloud explosion from the accidental release of flammable materials in a confined and congestion location. Design accidental blast loads shall be provided by LANL. Explosions result in blast overpressure, high velocity fragments, dynamic (blast wind) pressure, and ground shock effects on structures. Buildings that have large surface areas are most vulnerable to blast overpressure effects. Nonstructural components and non-building structures have less surface area and structural response is affected most greatly by wind and drag effects due to dynamic pressure and by high velocity fragments. Design of these components and structures shall consider these effects in accordance with TM 5-1300 [22] or other similar references.

2.1.2 Load Combinations

A. Load combinations for PC-1 and PC-2 building structures were presented in Section II.1.B. These load combinations also apply to the design of the supports and anchorage for PC-1 and PC-2 nonstructural components and to non-building structures. Where the design is in accordance with Strength Design (SD) or Load & Resistance Factor Design (LRFD), the load combinations in Table II-5 shall be used. Where the design is in accordance with Allowable Stress Design (ASD), the load combinations in Table II-6 shall be followed. Additional significant loads for systems and components may include start up and operating loads that affect the design of supports and anchorage. These loads shall be considered to be live loads in the load combination rules.

2.2 NONSTRUCTURAL COMPONENT DESIGN

A. This section primarily covers the design of supports and anchorage of nonstructural systems and components. This includes complete requirements for the seismic design of those supports and anchorage. In addition, the section does provide some information and requirements for the seismic design of the systems and components.

B. Supports and anchorage of PC-1 and PC-2 nonstructural systems and components shall be designed in accordance with the IBC [5]. Seismic design of these components shall follow Section 9.6 of ASCE 7 [11] considering the components to be in Seismic Design Category D. All architectural, mechanical, electrical, and other nonstructural components in structures shall be designed and constructed to resist the loads listed in the previous section along with the equivalent static seismic forces and relative seismic displacements as described in this section. The design and evaluation of support structures and architectural components and equipment shall consider their flexibility as well as their strength. The functional and physical interrelationship of components and their effect on each other shall be designed so that the failure of a PC-1 or PC-2 architectural, mechanical, or electrical component shall not cause the failure of a nearby PC-2, PC-3, or PC-4 mechanical or electrical component.

C. Components shall be attached such that component forces due to combined loads on the component are transferred to the structure. Component attachments for seismic loads shall be bolted, welded, or otherwise positively fastened without consideration of the frictional resistance produced by gravity. A continuous load path of sufficient strength and stiffness between the component and supporting structure shall be provided. Local elements of the structure shall be designed and constructed for the component forces where they control the design of the elements or their connections. The DBD shall provide sufficient information relating to component attachment to verify compliance with these requirements.

2.2.1 Seismic Forces

A. The seismic design force for nonstructural components is F_p as given by Equation II-18. This seismic design force is centered at the component's center of gravity and distributed relative to the component's mass distribution.

$$F_p = \frac{0.4 a_p S_{DS} W_p}{R_p / I_p}\left(1 + 2\frac{z}{h}\right) = \frac{0.22 a_p W_p}{R_p / I_p}\left(1 + 2\frac{z}{h}\right) \qquad \text{Eq. II-18a}$$

F_p is not required to be taken as greater than

$$F_p = 1.6 S_{DS} I_p W_p = 0.86 I_p W_p \qquad \text{Eq. II-18b}$$

and F_p shall not be taken as less than

$$F_p = 0.3 S_{DS} I_p W_p = 0.16 I_p W_p \qquad \text{Eq. II-18c}$$

where I_p is a component importance factor of:

- I_p = 1.0 for PC-1
- I_p = 1.5 for PC-2

B. The component importance factor, I_p, is used for PC-2 components that must have a higher level of assurance of function. Other parameters in Equation II-18 are:

- S_{DS} is 0.54g, the peak of the 5% damped design ground response spectrum.
- a_p is a component amplification factor that varies from 1.0 to 2.5.
- W_p is the component operating weight.
- R_p is a component response modification factor that varies from 1.5 to 5.
- z is the height in the structure of point of attachment of the component with respect to the base of the structure.
- h is the average roof height of the structure with respect to the base.

C. Alternately, seismic force, F_p, may be determined using Equation II-19 and the seismic response accelerations, a_i at level i of the structure, obtained from either modal response spectra seismic analyses in accordance with Section II.1.5.2 or response history seismic analyses as discussed in Section II.1.5.3.

$$F_p = \frac{a_i a_p W_p}{R_p / I_p} A_x \qquad \text{Eq. II-19}$$

where A_x is the torsional amplification factor determined from Equation 9.5.5.5.2 of ASCE
a_i is the seismic response acceleration at level i of the structure.

D. The component amplification factor, a_p, and the component response modification factor R_p, are determined from Table 9.6.2.2 of ASCE 7 for architectural components and Table 9.6.3.2 for mechanical and electrical components as a function of the characteristics of the nonstructural component. A summary of these factors is presented in Table II-9. More flexible components such as cantilever elements that are unbraced or braced to the structural frame below their center of mass have a_p of 2.5; while more rigid components such as cantilever elements that are braced to the structural frame above their center of mass have a_p of 1.0. Per Section 9.2.1 of ASCE 7, items that have their fundamental frequency less than 16.7 hz (0.06 second period) are considered flexible and items that have their fundamental frequency greater than 16.7 hz are considered rigid.

E. Components that are comprised of high deformability elements and attachments (welded steel piping or moment frames) have an R_p value of 3.5; components that are comprised of limited deformability elements and attachments (steel braced frames) have an R_p value of 2.5; components that are comprised of low deformability elements and attachments (cast iron) have an R_p value of 1.5. For distributions systems such as conduit and cable trays that are highly deformable, the R_p value is 5.0.

F. The force, F_p shall be applied independently longitudinally, and laterally in combination with other non-seismic concurrent loads associated with the component. *It is acceptable for simple systems (i.e. electrical cabinet, etc.) to apply the seismic horizontal load at the c.g. of the item. For more complicated systems (i.e. distribution systems) distribute the lateral force in proportion to the mass distribution of the system or component.* Vertical load effects are accounted for through the use of Equation II-3. Earthquake effects, E, to be used in the load combination equations are determined for nonstructural components by substituting F_p for Q_E in Equation II-3. The reliability/redundancy factor, ρ, is permitted to be taken equal to 1.

Table II-9 Component Amplification Factors, a_p & Response Modification Factors. R_p for Non-Structural Systems and Components

Architectural Component or Element	a_p	R_p
Interior Nonstructural Walls and Partitions		
Plain (unreinforced masonry walls)	1.0	1.5
All other walls and partitions	1.0	2.5
Cantilever Elements (Unbraced or braced to Structural Frame Below Its Center of Mass	2.5	2.5
Cantilever Elements (Braced to Structural Frame Above Its Center of Mass	1.0	2.5
Exterior Nonstructural Wall Elements and Connections		
Wall element and body of wall panel connection	1.0	2.5
Fasteners of the connecting system	1.25	1.0
Veneer	1.0	2.5
Penthouses (Except when Framed by an Extension of the Building Frame)	2.5	2.5
Ceilings	1.0	2.5
Storage Cabinets and Laboratory Equipment	1.0	2.5
Special Access Floors (ASCE 7 9.6.2.7.2)	1.0	2.5
All Other Access Floors	1.0	1.5
Other Rigid Components		
High deformability elements and attachments	1.0	3.5
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.5
Other Flexible Components		
High deformability elements and attachments	2.5	3.5
Limited deformability elements and attachments	2.5	2.5
Low deformability elements and attachments	2.5	1.5
Mechanical and Electrical Component or Element	a_p	R_p
General Mechanical Equipment		
Boilers and furnaces and others	1.0	2.5
Pressure vessels on skirts, free standing stacks, cantilevered chimneys	2.5	2.5
Manufacturing and Process Machinery	1.0	2.5
Piping Systems		
High deformability elements and attachments	1.0	3.5
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.5
HVAC Systems		
Vibration isolated	2.5	2.5
Nonvibration isolated, mounted in-line with duct work, and others	1.0	2.5
Elevator Components	1.0	2.5
Trussed Towers (free standing or guyed)	2.5	2.5
General Electrical		
Distribution systems (bus ducts, conduit, cable tray)	2.5	5.0
Equipment	1.0	2.5
Lighting Fixtures	1.0	1.5

G. Alternately, the seismic force acting on systems and components may be determined from in-structure response spectra at the system and component attachment point. In-structure response spectra are generally required to be generated for PC-3 or PC-4 building structures. Note that in-structure response spectra are available from the LANL Chapter 5 POC for many existing LANL structures. For PC-1 or PC-2 systems and components located in PC-3 or PC-4 buildings, the in-structure response spectra may be reduced by the ratio of peak ground accelerations for the evaluation of the systems and components. The peak ground accelerations for LANL PC-3 and PC-4 seismic design are 0.34 and 0.58g, respectively (Section III). The peak ground acceleration of PC-1 and PC-2 seismic design is $0.4S_{DS}$ or 0.4(0.54) = 0.22g.

2.2.2 Seismic Relative Displacements

A. When a nonstructural component (primarily, distribution systems) is supported at two points on the same structure or at two points on separate structures, seismic relative displacements place forces and stresses in the component in addition to the seismic inertial force, F_p. The maximum displacements of the support points, δ_x (at level x in the supporting structure) shall be estimated from the seismic analyses of the supporting structure using Equation II-10. If the resulting seismic relative displacements are judged to produce seismic response in the component that is significant when compared to seismic response due to seismic forces, F_p, forces, stresses, and deformations of the multiply-supported nonstructural component shall be determined by a static analysis of the component subjected to the relative support point displacements. The effect of relative seismic displacements shall be obtained by using the worst combination of peak displacements or by proper representation of the relative phasing characteristics associated with different support inputs. When the response due to seismic relative displacement is less than about 30% of the response due to seismic inertial forces, the contribution of the seismic relative displacement is judged to be insignificant.

B. Let the seismic response of the component due to seismic force, F_p in accordance with Equation II-3 be designated as E_F for the purpose of considering seismic relative displacement. Let the seismic response of the component subjected to seismic relative displacements as determined from the static displacement analysis be designated as E_δ. When the contribution from seismic response displacements is significant, it shall be combined with response from seismic inertial forces in accordance with the following equation.

H. $$E = \sqrt{E_F^2 + E_\delta^2} \qquad \text{Eqn. II-20}$$

C. The resulting seismic response, E shall be combined with response from non-seismic loads in accordance with the load combination equations presented previously in this section.

2.2.3 Component Anchorage

A. Components shall be anchored in accordance with the following provisions:

- The force in the connected part shall be determined based on the prescribed forces for the component as given by Equations II-18 or II-19. Where component anchorage is provided by shallow expansion anchors, shallow chemical anchors, or shallow (low deformability) cast-in-place anchors, a value of R_p = 1.5 shall be used in Equations II-18 or II-19 to determine the force in the connected part.
- Anchors embedded in concrete or masonry shall be proportioned to carry the least of the following:
  - The design strength of the connected part
  - 1.3 times the force in the connected part due to the prescribed forces
  - The maximum force that can be transferred to the connected part by the component structural system
- Determination of forces in anchors shall take into account the expected conditions of installation including eccentricities and prying effects.
- Determination of force distribution of multiple anchors at one location shall take into account the stiffness of the connected system and its ability to redistribute loads to other anchors in the group beyond yield.
- Steel base plates for supports resisting significant overturning moment in addition to axial and shear forces shall have no fewer than four anchor bolts. This requirement will assure that overturning moments are not resisted solely by prying action between the anchor bolts and the base plate.
- Powder driven fasteners shall not be used for tension load applications unless approved by the LANL Chapter 5 POC.
- The design strength of anchors in concrete shall be determined in accordance with the provisions of ACI 318.
  - Anchors should be designed to have sufficient embedment such that the capacity of the anchorage is controlled by the anchor bolt material and not the concrete or masonry to which it is attached.[16]
- For expansion anchors in concrete or masonry, capacities may be taken from vendor data provided that those capacities are supported by ICBO evaluation reports (or other independent test laboratory).
- The capacity of the welded connection shall be determined using the requirements of AISC-ASD [12] for ASD and AISC-LRFD [13] for SD.
- Anchorage to masonry structural elements may be designed by either ASD or SD measures. For ASD anchorage to masonry design:
  - Allowable loads for anchor bolts such as expansion anchors, toggle bolts, sleeve anchors, etc. (all anchors not solidly grouted into masonry) shall be determined by test as described in Section 2.1.4.1 of ACI 530 [17].[17]
  - Allowable loads for plate, headed, and bent bar anchor bolts shall be determined using the criteria described in Section 2.1.4.2 of ACI 530 [17], or by test as described in Section 2.1.4.1 of ACI 530 [17].

[16] This requirement provides for ductile anchorage that is good practice for earthquake design.

[17] Tests are performed per ASTM E 448 except that a minimum or five tests shall be conducted and the allowable load for meeting the ASD design loads may not be greater than the average of the tests divided by a factor of safety or five.

  - o Shear/tension interaction shall be linear as described in Section 2.1.4.2.4 of ACI 530 [17].
- For SD anchorage to masonry design:
  - o There is no SD criteria for determining allowable loads for anchor bolts such as expansion anchors, toggle bolts, sleeve anchors, etc. (all anchors not solidly grouted into masonry). Therefore, these anchorages shall be designed as described above for ASD.
  - o Allowable loads for plate, headed, and bent bar anchor bolts shall be determined using the criteria described in Section 3.1.6 of ACI 530 [17], using the understrength, Φ, factors described in Section 3.1.4.4 of ACI 530 [17].
  - o Shear/tension interaction shall be linear as described in Section 3.1.6.4 of ACI 530 [17].

2.2.4 Component Design Provisions

A. Architectural components shall be designed in accordance with the Seismic Design Category D provisions of Section 9.6.2 of ASCE 7 [11]. Mechanical and electrical components shall be designed in accordance with the Seismic Design Category D provisions of Section 9.6.3 of ASCE 7 [11].

B. Architectural component design provisions in ASCE 7 Section 9.6.2 address exterior nonstructural wall elements, interior nonstructural walls and partitions, parapets, chimneys and stacks, veneer, penthouses, suspended ceilings, storage cabinets and racks, laboratory equipment, access floors, appendages and ornamentation, and glass in glazed curtain walls, glazed storefronts, and glazed partitions. Important design concepts for architectural components are to assure that these components are adequately fastened to the structure and to assure that, when fastened, the components can accommodate story drifts of the supporting structure due to earthquake ground shaking.

C. Mechanical and electrical component design provisions in ASCE 7 Section 9.6.3 address boilers and furnaces, pressure vessels on skirts, manufacturing and process machinery, conveyors, piping systems, HVAC systems, elevator and escalator components, trussed towers, electrical distribution systems, and equipment, and lighting fixtures. Mechanical and electrical component supports and attachments include structural members, braces, frames, skirts, legs, saddles, pedestals, cables, guys, snubbers, and tethers as well as elements forged or cast as a part of the mechanical or electrical component. Important design concepts for mechanical and electrical components are to assure that there is a complete load path for component supports, that the load path is sufficiently stiff that it does not impair the operations of the component, and that mechanical and electrical component supports are adequately attached to the supporting structure. In addition, distribution systems at the interface of adjacent structures or portions of the same structure that may move independently shall be provided with adequate flexibility to accommodate the anticipated differential movements between the portions that move independently.

D. Fire protection sprinkler systems designed and constructed in accordance with NFPA 13, *Standard for the Installation of Sprinkler Systems,* must also be shown to meet the seismic force and seismic relative displacement requirements of this section. PC-2 mechanical and electrical components that are required to by safety analyses remain operable after the design earthquake shall demonstrate operability by shake table testing or experience data. Alternately, such components may demonstrate operability by rigorous stress and deformation analysis. Such components shall meet the inspection and certification requirements of Section A.9.3.4.5 of ASCE 7 [11].

E. The following additional restrictions apply to the evaluation of ceilings and access floors:

1. Ceilings weight shall include all light fixtures and other equipment or partitions that are laterally supported by the ceiling. For purposes of determining the seismic force, use a ceiling weight of not less than 4 pounds per square foot
2. Ceilings constructed of lath and plaster or gypsum board screw or nail attached to suspended members that support a ceiling at one level extending from wall to wall need not be analyzed provided the walls are not over 50 feet apart.
3. W_p (Component Operating Weight) for access floor systems shall be the dead load of the access floor system plus 25 percent of the floor live load plus a 10 psf partition load allowance.
4. Independently support light fixtures and mechanical services installed in metal suspension systems for acoustical tile and lay-in panel ceilings from the structure above.[18]

F. Cranes and hosts are to be designed to the criteria specified in AISC-ASD [12] and AISC-LRFD [13] for the load combinations given is Section II.1.2. The live load should include the maximum hoist load including the standard impact factors in AISC except when earthquake loads are considered.[19] LANL has developed a standard specification for vendors who provide bridge cranes at the LANL site. S*pecification, Section 14645 – Bridge Cranes to be edited to suit the particular project.*

2.2.5 Seismic Qualification of Nonstructural Components

A. For PC-1 and PC-2 nonstructural components, the seismic design and evaluation shall be based on the total lateral seismic force, F_p, and seismic relative displacements as described in Sections II.2.2.1 and II.2.2.2. For PC-2 components required by safety analysis to remain functional during or after an earthquake, shake table testing or experience data shall generally be used for seismic qualification. In some cases, it may be possible to demonstrate that a component can function following an earthquake by detailed dynamic analysis.

[18] This is good practice to prevent the falling of these items during an earthquake on personnel working below.
[19] It is not necessary to design PC-1 and PC-2 cranes or hoists for earthquake loads while loaded because the crane or hoist is only used a small fraction of the time and therefore, it is unlikely that it would be used during a design earthquake and the reduced safety consequences for PC-1 and PC-2 SSC.

B. For qualification by shake table testing, experience data, or dynamic analysis, it is necessary to have an in-structure response spectrum at the component support point within the building structure. For PC-1 and PC-2 nonstructural components, the 5% damped in-structure response spectrum is used. If dynamic analyses of the building structure are performed, in-structure response spectra at component support points can be evaluated by methods described in ASCE 4 [6]. If in-structure response spectra are not available, spectra may be developed from the total lateral seismic force, F_p, using the approach given in ICBO AC156 [47].

C. Seismic qualification of PC-1 and PC-2 supports and anchorage of nonstructural components by analysis shall be conducted using equivalent-static analysis, linear response spectra analysis or time history analysis as presented for building structures in Sections II.1.3, II.1.4, II.1.5, and II.1.6. Regardless of the procedure followed, it is important that:

- The input to the component shall be defined either by an equivalent static lateral seismic force, F_p, in-structure response spectrum, or a response spectrum compatible acceleration time history.
- A load path evaluation for seismic induced inertial forces shall be performed. A continuous load path, or paths, with adequate strength and stiffness shall be provided to transfer all forces from the point of application to the support point on the structure.
- Seismic evaluation shall be conducted for the two orthogonal horizontal components of earthquake motion with responses combined by SRSS. The vertical component is included with the load combinations in Section II.1.2. In general, the orthogonal axes shall be aligned with the principal axes of the structure.

D. Mechanical or electrical systems and components may be qualified by testing or testing and analysis, following the guidance of IEEE-344 [16], ASME-QME-1 [9], or ICBO AC156 [47], as applicable. The required response spectrum (RRS) must exceed the in-structure spectra by:

RRS ◂ 1.1 (in-structure spectra) for PC-1 and PC-2.

E. The TRS of the test table motion must envelop the RRS at all frequencies.

2.3 NON-BUILDING STRUCTURE DESIGN

A. Non-building structures shall be designed in accordance with the IBC [5]. Seismic design of these structures shall follow Section 9.14 of ASCE 7 [11] considering the structures to be in Seismic Design Category D. All non-building structures shall be designed and constructed to resist the loads listed in Section II.2.1 along with seismic loads as described in this section. The design of non-building structures shall provide adequate stiffness, strength, and ductility consistent with the requirements specified in Section II.1.0 for buildings.

B. Non-building structures having industry standards for their structural design shall be designed in accordance with those standards. Industry standards for several non-building structures are listed in Table 9.14.3 of ASCE 7 [11]. In addition, non-building structures shall be designed for the minimum seismic lateral forces presented herein. Non-building structures shall be designed for the seismic base shear in a given horizontal direction in accordance with:

$$V = C_s W = \frac{S_{DS}}{R/I_E} W = \frac{0.54W}{R/I_E} \qquad \text{Eq. II-21a}$$

The seismic base shear need not be greater than:

$$V = C_s W = \frac{S_{D1}}{T(R/I_E)} W = \frac{0.26W}{T(R/I_E)} \qquad \text{Eq. II-21b}$$

The seismic base shear shall not be taken less than:

$$V = C_s W = 0.044 S_{DS} I_E W = 0.024 I_E W \qquad \text{Eq. II-21c}$$

where S_{DS} is 0.54g and S_{D1} is 0.26g as specified in Section II.1.1, W is the effective seismic weight of the non-building structure, I_E is the importance factor for the non-building structure of 1.0 for PC-1 and 1.5 for PC-2, and T is the fundamental period of the non-building structure in the direction being considered.

C. R is a response modification coefficient that is taken to be the lesser of the values given in ASCE 7 Table 9.14.5.1.1 for non-building structures and ASCE 7 Table 9.5.2.2 for building structures. Overstrength factor, Ω_o, shall be taken from the same table as the R factor. For non-building structures that have an R value taken from ASCE 7 Table 9.14.5.1.1, Equation II-21c shall be replaced with:

$$V = C_s W = 0.14 S_{DS} I_E W = 0.076 I_E W \qquad \text{Eq. II-21d}$$

D. Vertical distribution of the lateral seismic forces in non-building structures shall be determined by:

- The equations of ASCE 7 Section 9.5.5.4
- Modal seismic analysis of the non-building structure (ASCE 7 Section 9.5.6)
- In accordance with a standard approved by the LANL Chapter 5 POC and applicable to the specific non-building structure.

E. For tanks, vessels, bins, silos, and similar containers of liquids, gases, or granular solids supported at the base, the minimum seismic design force shall not be less than that required by the approved standard for the specific system.

F. Irregular non-building structures per ASCE 7 Section 9.5.2.3 that cannot be modeled as a single mass shall be evaluated by modal analyses methods as presented in ASCE Section 9.5.6 and Section II.1.5.2 of this ESM chapter.

G. When an approved national standard provides a basis for the earthquake resistant design of a particular type of non-building structure covered in ASCE 7 Section 9.14, the use of such a standard is subject to the following limitations:

- The input seismic ground motion shall be in conformance with Section II.1.1.10.

- The values for total lateral force and total base overturning moment shall not be less than 80 percent of the base shear value and overturning moment, each adjusted for the effects of soil-structure interaction that is obtained using Equation II-21.

H. The base shear is permitted to be reduced in accordance with ASCE 7 Section 9.5.9.2.1 to account for the effects of soil-structure interaction. In no case shall the reduced base shear be less than 70 percent of that obtained from Equation II-21.

I. Rigid non-building structures (fundamental frequency greater than 16.7 hz) shall be designed for the lateral force obtained from:

$$V = 0.30 S_{DS} I_E W = 0.16 I_E W \qquad \text{Eq. II-22}$$

J. This force shall be distributed with height in accordance with ASCE 7 Section 9.5.5.4.

K. The weight, W, for non-building structures shall include the dead load as defined for buildings. For purposes of calculating design seismic forces in non-building structures, W also shall include all normal operating contents for items such as tanks, vessels, bins, hoppers, and the contents of piping. W shall include snow and ice loads when those loads constitute 25 percent or more of W or when required by LANL.[20]

L. The fundamental period of the non-building structure shall be determined by methods as prescribed in ASCE 7 Section 9.5.5.3 for buildings or by other rational methods.

M. The drift limitations of ESM Section II.1.6.6 need not apply to non-building structures if a rational analysis indicates that they can be exceeded without adversely affecting structural stability or attached or interconnected components and elements such as walkways and piping. P-delta effects shall be considered when critical to the function or stability of the non-building structure.

N. Specific design requirements for non-building structures are provided in ASCE 7 Sections 9.14.6 and 9.14.7. These provisions shall be followed for LANL PC-1 and PC-2 non-building structures.

O. Section 9.14.6 considers non-building structures that are similar to buildings. These structures include:

- Pipe racks
- Steel storage racks
- Electrical power generating facilities
- Structural towers for tanks and vessels
- Piers and wharves (not applicable to LANL)

P. Section 9.14.7 considers non-building structures that are not similar to buildings. These structures include:

- Earth retaining structures
- Tanks and vessels
- Stacks and chimneys

[20] For large non-building structures, less than 100 percent of the snow and ice loads may be justified. For example, for building structures W includes only 20% of the flat snow load where that load exceeds 30 psf.

- Amusement structures (not applicable to LANL)
- Special hydraulic structures (e.g., separation walls, baffle walls, weirs, etc.)
- Secondary containment systems (e.g., impoundment dikes and walls)
- Telecommunication towers

Q. Modeling, analysis, and design of non-building structures shall follow the provisions for buildings as presented in Sections II.1.3, II.1.4, II.1.5, and II.1.6, with exceptions as noted in this section.